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4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

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Note.—Part 2 of this Journal is the 1960-42 Newsletter of the Highway Division.

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L T S DESIGN OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENT[a]

By Benjamin F. McCullough[1] and William B. Ledbetter[2] A.M. ASCE

SYNOPSIS

With the advent of the continuously reinforced concrete pavement, without joints, the highway engineer has been forced to discard existing design theories and rely largely on experimentation and judgment to attempt to properly design this new type of pavement.

This paper is aimed at presenting a rational and uncomplicated approach to the approximate design of continuously reinforced concrete pavement, considering all of the factors and variables that can be evaluated by means of the engineering tools available. The two major factors to be considered, internal forces developed from restrained pavement volume changes and external forces developed from the traffic loads, are examined, and methods are given whereby each can be evaluated in arriving at an economically safe design. The Load-Temperature-Shrinkage design approach is predicted on the assumption that the concrete should be designed to withstand the external forces developed from traffic loads, and the reinforcing steel should be designed to withstand the internal forces developed from restrained pavement volume changes. It is shown that a pavement thickness of 7.0 in., with 0.5% interior longitudinal hard grade steel over a subgrade whose modulus of subgrade reaction is 100 psi per in., will be sufficient to carry a 16,000 lb design wheel load. Past experiences are cited to indicate that this design approach is adequate. In addition,

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[a] The opinions expressed in this paper are exclusively those of the authors and do not necessarily reflect the thinking or the policy of the Texas Highway Department.

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the areas for possible modification of the approach by experimentation and research are examined.

GENERAL BACKGROUND

The first extensive concrete road system was constructed in Wayne County, Michigan, in 1909. Since that time, the highway engineer has been faced with the problem of providing an adequate pavement structure that will carry the traffic load and withstand the phenomenon of concrete volumetric changes. The solution to this problem has been a process of evolution.

To control volumetric changes, the first designs provided for expansion joints at frequent intervals. At these locations, all of the expansion and contraction of the concrete was to occur, because the subgrade friction restrained volumetric change and random cracking occurred between the expansion joints. This proved to be unsightly and troublesome. To remedy this, the distance between expansion joints was increased, and failure planes were installed at intervals between these expansion joints. These spacings were considered as the natural crack spacing due to contraction. The next design step was to install sufficient reinforcement in the pavement slab to drag both ends of the individual slab over the subgrade toward its center. This still involved the use of contraction or load transfer joints every 100 ft, because all longitudinal, horizontal movement due to volumetric change was assumed to occur at the joints.

Every design had incorporated expansion and contraction joints or a combination of the two. Although highway engineers seldom have a meeting of minds on problems of pavement design, there is practical unanimity in the opinion that the use of joints leaves much to be desired. Joints allow detrimental water seepage, have an adverse effect upon the riding qualities and present a maintenance problem.

In recent years, a new concept of concrete design has been introduced. This concept is based on the premise that joints can be eliminated by the introduction of moderate amounts of reinforcing steel in such quantities to force the concrete to develop numerous transverse, hairline cracks. The steel does not prevent cracking. On the contrary, it induces cracking. However, it keeps the cracks tightly closed. This is very advantageous because the hairline cracks do no damage as the crack width is sufficiently small to (1) prevent the passage of water from the surface to the subgrade, and (2) insure sufficient aggregate interlock to prevent extreme shearing forces in the steel due to traffic loads. Several state highway departments are currently experimenting with this type of pavement with excellent results. States such as Indiana, Illinois, Texas, California, New Jersey, and Pennsylvania are trying this idea and other states are considering it.

Stress Producing Elements.—The design of a continuously reinforced concrete pavement depends on the proper correlation of many variables and unknowns, the more important of which are the internal and external forces that produce stresses in the concrete and steel. As evidenced by the problem of cracking in concrete, the tensile aspects of these forces acting on the relatively weak tensile strength of concrete become the major factor in the proper design of continuously reinforced concrete pavement.

Internal Forces.—The internal forces produced in a continuously reinforced concrete pavement are the result of restrained concrete volumetric changes. The two prime factors causing volumetric change in reinforced concrete are shrinkage phenomena and temperature differential.

Shrinkage.—Shrinkage of concrete is, for the most part, a result of the consolidation of mass that takes place as moisture is lost during the hydration process. This volume reduction occurs in three dimensions and is not uniform throughout the mass. The major portion of the shrinkage occurs in the transverse cross section in which the concrete is relatively free to contract. Longitudinal shrinkage is opposed by subbase friction and by the longitudinal steel through the developing bond (1).[3] The development of bond between the concrete and steel is largely the result of shrinkage of the concrete around the steel, therefore, it will have the same development rate as shrinkage (2). The full development of bond consists of the distinct stages (2): of (1) the adhesive stage in which the materials deform without relative movement; (2) the frictional stage in which there is relative movement between the two materials and the resistance is frictional; and (3) the bearing stage in which the roughness projections of the bar press against the concrete.

Since bond strength is a direct result of shrinkage, non-uniform shrinkage causes a pattern consisting of areas of higher or lower bond strengths. As the curing process continues, longitudinal shrinkage flow is towards the more effectively bonded areas and away from the less effectively bonded areas. The more effectively bonded areas experience the full effects of stage one, whereas, the less effectively bonded areas depend primarily on the final two stages, because only partial benefit is derived from the first stage. This alternating pattern of bond effectiveness subjects the concrete between the areas of more effective bond to a pulling or internal tensile force. When this tensile force exceeds the tensile strength of the concrete, a crack is formed.

Between the cracks, the concrete will attempt to shrink further, but is partially held back by the reinforcement. Referring to Fig. 1, it can be seen that the concrete of length AB would contract back to position A_1B_1 (a state of zero internal stress) if free shrinkage occurred. However, the resistance offered by the steel through bond restrains movement and allows only partial shrinkage back to a position that is less than the free shrinkage position (position A_2B_2). As a result of this restraint, at the mid-portion of the crack interval the concrete will be in tension and the steel will be in compression (see Fig. 2). Because the overall length of the steel remains unchanged, the steel must be in tension near the crack to offset the compression strains at the center. In other words, the total shortening due to compression must be equal to the total lengthening due to tension (3).

Temperature Differential.—A volumetric change due to a temperature differential places an internal stress in both the concrete and the steel, whereas, shrinkage produces an internal stress in the concrete only, that, in turn, is transferred to the steel through bond. Because a temperature drop produces the critical tensile forces, only this phase of the temperature differential will be considered.

Due to the relatively equal magnitudes of thermal coefficients for steel and concrete, a temperature drop causes similar contractions in both materials. However, because a long continuously reinforced concrete slab is effectively

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[3] Numerals in parenthesis—thus; (1)—refer to corresponding items in the Appendix Bibliography.

restrained against free contraction by subgrade friction, tensile stresses are built up in the slab in areas between the cracks. If the internal stress in this area exceeds the tensile strength of the concrete an additional crack is formed. If there is an insufficient quantity of steel at any crack, the steel stresses, resulting from forces caused by temperature changes, may stress the steel beyond its yield point and cause a pavement failure.

For the purpose of examining the stress variations in the two materials, a free body between the cracks is selected (see Fig. 3). If the effect of the concrete on the steel through bond is ignored, the restrained steel would have a stress diagram similar to that shown in the top diagram in Fig. 4. Because the steel is restraining the concrete from contracting freely towards its center through bond, the concrete is placed in tension. This results in the transfer of compressive stresses to the steel through bond in the area between cracks (see Fig. 4).

The resultant stress diagram for the steel (see Fig. 4) would have the greatest tensile stress near the crack with a smaller stress near the mid-

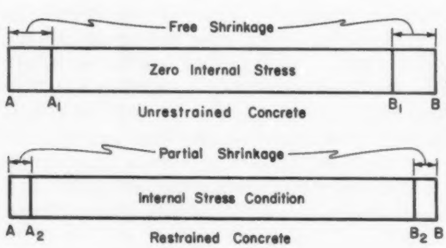

FIG. 1.—EFFECT OF SHRINKAGE ON A PORTION OF CONCRETE PAVEMENT BETWEEN CRACKS

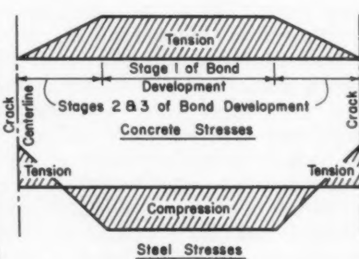

FIG. 2.—STRESS DISTRIBUTION BETWEEN CRACKS IN A CONCRETE PAVEMENT FOR AN INTERNAL STRESS CONDITION RESULTING FROM RESTRAINED SHRINKAGE

portion. There is a progressive increase in bond development from the crack to some point in the mid-portion of the slab. Therefore, compressive forces are transferred to the steel that oppose the tensile forces developed in the steel due to temperature drop. This results in a smaller steel stress near the mid-section than at the crack. This concept is borne out in recent tests that showed the stress of the steel at the crack was of considerably greater magnitude than the interior portion (4).

The resultant stress diagram for the concrete (see Fig. 4) would have zero stress at the crack with a progressive increase in tension towards the center as the steel resists the contraction through the final two stages of bond development. Full tension in the concrete would be developed in the mid-portion at which there is no relative movement between the concrete and the steel.

It should be emphasized here that in order to preclude overstressing the steel at the crack due to stresses developed by the restrained volume changes, sufficient longitudinal steel should be placed to insure the concrete forming another crack rather than stressing the steel beyond yield.

External Forces.—The two most important external force producing agents on concrete pavement are wheel loads and differential subgrade swell due to moisture variation. Because a properly constructed subgrade should eliminate most of the major swell, this characteristic will not be considered in this report.

The wheel load produces a critical flexural action in the section of the slab between the cracks. This force results in tensile and compressive stress in the bottom and top, respectively, of that part of the slab directly beneath the

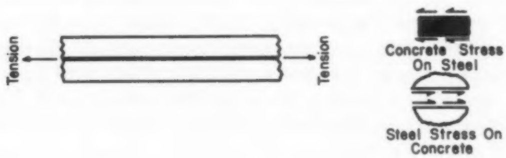

FIG. 3.—STRESS VARIATIONS BETWEEN CONCRETE AND STEEL IN A PAVEMENT RESULTING FROM TEMPERATURE STRESSES

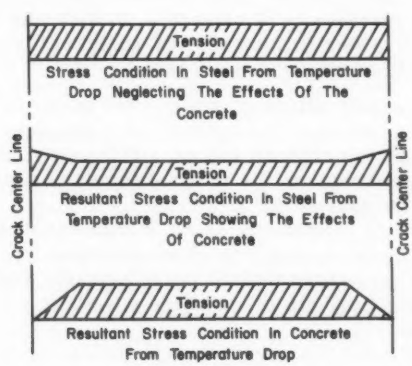

FIG. 4.—STRESS CONDITION IN THE STEEL AND THE CONCRETE RESULTING FROM TEMPERATURE STRESSES

wheel load and the opposite reaction some distance away (5). These stresses superimposed on the stresses due to the internal forces may result in additional cracks if the tensile strength of the concrete is exceeded. In the formation of a pavement crack pattern experience has indicated that the internal forces may be the main factor in their development, with the external forces produced from traffic loads having only a minor effect (6),(7).

At the crack the wheel load can develop a shearing force of considerable magnitude. Steel breaks on the Illinois pavement after approximately 9 yr service indicates that the primary breaking force was the shearing developed by

the heavy wheel loads (8). If the design provides for holding the cracks tightly closed, this condition should not occur, because the aggregate interlock will prevent the condition in which shearing stresses are developed.

From the discussion of the stress producing elements, it is seen that the final crack pattern is a result of the internal and external forces acting on a slab over a period of time. But the initial shrinkage, occurring while the concrete is still relatively "weak" in tensile strength, causes the majority of the initial cracks and establishes the primary crack pattern due to the stress concentrations it sets up.

The criteria of the proposed design procedure for continuously reinforced concrete pavement should provide a method for evaluating the magnitude of stresses produced by the internal and external forces in order to determine a percentage of steel and a slab depth for the design conditions. From an objective viewpoint, the goal of a proposed design should (1) provide sufficient steel to develop new cracks for stress relief, rather than the existing cracks opening wider or causing steel yield at the cracks; and (2) determine the depth of the slab based on both the steel and the concrete carrying the stresses developed by the external forces.

DESIGN PROCEDURE AND DEVELOPMENT

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in Appendix III.

Assumptions.—In order to simplify the analytical approach to the design method, the following assumptions are made:

1. For initial considerations of design, the steel and concrete are considered to act independently. The concrete is considered to totally carry the stresses produced by the external forces, and the steel is considered to control the stresses produced by the internal forces resulting from restrained volume changes in the pavement.
2. The concrete is considered to be a homogeneous, isotropic, elastic solid in equilibrium, over a subgrade whose reactions are vertical.
3. Transverse cracks have formed in the concrete, and there is no volume change in the longitudinal direction of the pavement between these cracks, that is, full restraint.
4. There is sufficient distance between cracks for bond to fully develop between concrete and steel.
5. The thermal coefficient of steel and concrete are the same.
6. The total forces developed by the restraint from volume changes are carried by the concrete and the steel only.
7. Bond stresses are constant.

Design Procedure.—

Determination of Trial Thickness.—By first temporarily disregarding the effect of the steel for determination of a required thickness of pavement, existing standard methods of pavement design can be utilized. The one selected for use in designing continuous pavement is that of H. M. Westergaard (9). The first consideration is the interior or center portions of the pavement. Because there are no edges or corners in the center portion, Westergaard's interior equation can be used. Westergaard's theoretical equation for interior

stresses S_i due to load is

$$S_i = \frac{0.3162\ W}{d^2}\left[4 \log_{10} L/b + 1.069\right] \quad \ldots\ldots\ldots\ldots (1)$$

in which W is the load, in pounds, d denotes the trial thickness, in inches, and in which

$$L = \sqrt[4]{\frac{E\ d^3}{12\left(1 - \mu^2\right)K}} \quad \ldots\ldots\ldots\ldots\ldots (2)$$

and

$$b = \sqrt{1.6\,a^2 + d^2} - 0.675\,d \text{ (when } a < 1.724\,d)$$

Or

$$b = a \text{ (when } a \geqq 1.724\ d)$$

In computing the required minimum trial thickness, a standard recommended practice in jointed pavement is to use a safety factor of 2, due to the number of repititions of load causing fatigue of the concrete; therefore, a safety factor of 2 is recommended for use in continuously reinforced concrete pavement design for interior stresses. Westergäard's theoretical equation is used in lieu of his modified equation for several reasons. First, present knowledge has not indicated any modification to be fully justified under all conditions. In 1939 Kelley (10), stated,

> "In the light of present knowledge it will be conservative and not uneconomical to continue to use the results given by the original Westergaard Analysis."

Second, Westergaard's modification of his original analysis was developed from experimental results of actual tests. These test results were from "jointed" pavement. However, a continuous pavement without joints certainly acts differently than jointed pavement and thereby causes doubt as to the applicability of these experimental modifications with continuous pavement. Third, deflection and strain tests were run on concrete slabs in the laboratory, and it was reported in the conclusion that for the interior portions of the pavement, values derived from Westergaard's theoretical formula correlate closely with the actual test values (11), (12). Fourth, with the advent of any new concept to a problem, advancements should be conservatively approached, keeping in mind that as experimental data develops the original approach may be modified to yield more efficient and economical results. To aid in designing for pavement thicknesses, Appendix I contains design charts for computing required interior thicknesses, using Westergaard's theoretical Eq. 1 for E varying from 1,000,000 psi to 4,000,000 psi.

Determination of Steel Ratio to Control Volume Changes.—The next step in the design is the determination of the minimum steel ratio, P, that will adequately control the volumetric changes in the concrete. The key word here is "control." The entire elimination of cracks is prohibitive in cost, if not impossible to accomplish, therefore, the economical approach is to place enough steel to "control" crack width, keeping the cracks numerous and relatively small in width and thereby forcing the concrete to act as a continuous unit. In 1933, Vetter (3) proposed a series of formulas to determine the (1) minimum steel ratio, P, to effectively control cracking and (2) the crack spacing, X, that would result from a given P. By disregarding the effects of the load momentarily, Vetter's analysis can be used to determine the minimum adequate amount of steel.

The minimum reinforcement necessary to control volume changes is governed by the yield tensile strength of the steel, S_s, and the tensile strength of the concrete S_c'. In order to prevent yielding of the steel before the concrete cracks from forces developed from restrained volume changes, the minimum steel ratio P is

$$P = \frac{S_c'}{S_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Under most circumstances, this minimum, P, will govern the amount needed for shrinkage. However, this minimum, P, should always be equal to or greater than

$$P \geqq \frac{S_c'}{S_s + z\,E_s - n\,S_c'} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which z is the shrinkage coefficient, and $n = \frac{E_s}{E_c}$. To control temperature changes the minimum steel ratio is

$$P = \frac{S_c'}{S_s - n\,S_c'} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

So long as

$$\text{Min} \quad P \geqq \frac{S_c'}{2\left(S_s - T\,\epsilon\,E_s\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

in which T is the total temperature drop in degrees Fahrenheit, and ϵ refers to the thermal coefficient.

Vetter's analysis discloses an interesting development. The amount of steel necessary to control both temperature and shrinkage forces simultaneously will always be less than the amount of steel needed to control either temperature or shrinkage alone. Vetter states:

> "It seems paradoxical that the addition of shrinkage to an existing temperature drop should reduce the steel tension. It must be remembered, however, that Eq. 24 (corresponds to Eq. 5 of this paper) expresses an equilibrium in which the concrete between the cracks is just on the point of breaking. If shrinkage is added, the unit will break into shorter units in which the concrete stress is again equal to the tension strength, but the steel tension corresponding to this stress distribution is less than for temperature drop alone."

Therefore, by determining the greatest amount of steel needed to satisfy Eqs. 3, 4, 5, and 6 we have determined the minimum needed to control volume changes.

From the foregoing, it is seen that the dominating factors in the determination of the required steel ratio are the tensile strength of the concrete and the yield strength of the steel. In general, the lower the strength of the concrete, the less the amount of required steel to control volume changes. This concept is better understood when you consider that if the concrete strength is increased, increased forces will be transferred to the steel through bond between cracks that are further apart. This results in more steel necessary to adequately carry the internal forces. Hence, Eq. 3 gives the absolute minimum steel necessary. In using Vetter's analysis, as in using Westergaard's theoretical equation, we remain on the conservative side. Some of these forces

undoubtedly will be transferred to the subgrade, depending on the subgrade's strength and the friction factor between the concrete and the subgrade. Also, some movement will take place between the steel and concrete, relieving to a certain degree some effect of these forces. Also, a large portion of the shrinkage due to hydration usually occurs before the full strength of the concrete is reached, forming cracks in the concrete that limit the magnitude of the detrimental temperature stresses. But, as was mentioned earlier, the inclusion of these variables in the light of present understanding would yield, at best, only problematic results. By approaching the problem on the safe side, a conservative beginning is effected whereby future experimental data can be compared toward corroborating or modifying the theoretical conservative approach.

The theoretical crack spacing, X, can be computed as follows:

$$X = \frac{(S'_c)^2}{n P^2 q u\left[(\epsilon T + z) E_c - S'_c\right]} \quad \ldots\ldots\ldots\ldots\ (7)$$

in which q is the ratio of perimeter of bars to area, and u denotes the unit bond stress in pounds per square inch. The crack spacing determined from this equation considers only the forces acting on the concrete and the steel from restrained volume changes and assumes a constant strength-concrete throughout. With the addition of loads other cracks undoubtedly will form and, if the concrete is weaker or stronger than the S'_c used in the equation, the crack spacing will vary accordingly. However, the equation does give a fairly good approximate indication of the crack spacing due to forces developed from restrained volume changes.

Tensile Properties of Concrete.—Because the main factor in the determination of the amount of steel needed to control volume changes is the tensile strength of concrete, this property should be discussed further here in order that a common ground can be reached on the determination of tensile strength for design purposes. Investigation reveals wide variation in published results on tensile strengths of concrete. To date, several methods have been devised to measure tensile strength but none have been adopted as a standard procedure. All available data was reviewed to determine what relationship, if any, did exist between tensile strength and flexural strength (modulus of rupture) of concrete using conventional gravel or crushed stone aggregates, in order that existing specifications calling for minimum acceptable flexural strengths could continue to be used for continuously reinforced pavement. Results show that the ratios of tension to flexure varied from 0.4 to 0.65, depending on the type and gradation of the aggregate, strength of the concrete, cement factor, water factor, and methods used for determination of tensile strength. Further research is needed to fully determine this relationship. Until more data can be accumulated, it is recommended that tests be made with the aggregates and cements to be used on the project in order to determine the relationship between tensile strength and flexural strength for design purposes.

Determination of Modified Thickness of Pavement.—The question of the load carrying capacity of the steel is all that remains to be developed as far as the interior portion of the pavement is concerned. The full answer is, of course, not known. Theories have been developed and ideas proposed, but they all have to fall back on trial and error for basis and experimental development for acceptance. The theory proposed here is that the steel will carry a percentage of the load in proportion to its "equivalent area" of concrete A'_c.

$$A'_c = n\ A_s \qquad (8)$$

This idea has been utilized in the design of reinforced concrete structural members since about the turn of the century and is based on the theory of flexure (13b).

> "The unit stress on any fiber of a homogeneous beam a given distance from the neutral axis is the same as the unit stress on any other fiber at the same distance from the neutral axis."

By taking a unit width of pavement we can compute the additional thickness d' present from the steel from

$$d' = P\ n\ d \qquad (9)$$

Then, the trial thickness d can be reduced by the amount d' to give the actual design thickness h.

$$h = d - d' \qquad (10)$$

This transformation of steel to equivalent area of concrete is an elastic one, limiting the stresses to the elastic limits of the material. This is again on the conservative side, but will serve as a reasonable basis for experimental development toward further understanding of the load carrying capacity of the steel embedded in concrete pavement.

Determination of Required Longitudinal Edge Strengthening.—The last logical step in the main design is to investigate the pavement stress conditions along the longitudinal edges of the pavement. Returning to Westergaard, and momentarily disregarding the steel again, his edge loading analysis will give a desired longitudinal edge trial thickness d_e.

$$S_e = \frac{0.572\ W}{d_e^2}\left[4\ \log_{10}\ L/b + 0.359\right] \qquad (11)$$

in which S_e is the maximum allowable edge stress in pounds per square inch.

In considering a reasonable value for S_e, logic indicates that a safety factor less than two might well be used because, obviously, fewer repetitions of loads occur near the edges of pavements as occur in the center portions. Investigation reveals very little actual information dealing with lateral placement of vehicles in relationship to percentages driving on the pavement edges. However, Taragin in a study (14), of vehicle placement as related to shoulder investigations found that an average of only 8.5% of all commercial vehicle traffic encroached on the shoulder of standard width pavements. By entering the Portland Cement Association (PCA) chart (15) showing the safety factor needed for the number of desired repetitions on concrete pavement, a safety factor of 1.5 is indicated for 8.5% of the repetitions. Therefore, until further information is made available, it is recommended that a safety factor of 1.5 be utilized in determining the maximum allowable working stress S_e along the pavement edges. By determining the edge thickness d_e required, and then obtaining the additional thickness, if any, required over the interior trial thickness d, an additional steel ratio P' can be determined by Eq. 9 for each edge, or the pavement can be thickened the appropriate amount according to current practices. When placing additional steel in the edges, it is assumed in this paper that only the edge 12 in. of pavement will be stressed more than the interior portion of the pavement. To aid in the design, Appendix I contains de-

sign charts for computing required edge thickness from Westergaard's Eq. 11 for "E" varying from 1,000,000 psi to 4,000,000 psi (Fig. 11).

Development of Design Procedure.—

Typical Design.—A typical design is developed to give an example of the extent of concrete and steel required to adequately carry the internal and external forces. The maximum equivalent design wheel load encountered on United States highways today is in the neighborhood of 16,000 lb. Current practice is to increase this design wheel load by 25% to allow for impact forces developed, that yields, for the most critical highways, a design load of 20,000 lb. Typical conditions found are subgrades whose modulus of reactions are 100 lb per cu in. and mean temperature differentials in the neighborhood of 70° F. Typical concrete and steel properties are as follows: Concrete 7 day modulus of rupture = 650 psi; E_c = 4,000,000 psi; μ = 0.15; ϵ = .000006 in. per in. per °F; z = .0004 in. per in.; S_c' = 260 psi; E_s = 30,000,000 psi; S_s = 40,000 psi (intermediate grade).

Following through the design procedure with this given data yields a design calling for a pavement thickness of 7.64 in., with minimum longitudinal steel percentages of 0.68% in the interior and 1.24% in the edge 12 in. of pavement (or an increase in the edge thickness of pavement of 0.35 in.). This is generally in line with current (1960) designs of existing continuously reinforced pavement in the United States.

Comparisons of Designs Varying the Type of Steel and the Concrete Strength.—Fig. 5 shows a comparison of designs using intermediate, hard, and welded wire fabric steels and design loads of 20,000 lb and 16,000 lb for various concrete flexural strengths over a subgrade modulus of 100 lb per cu in. It is of interest to note that only 0.50% longitudinal steel is called for with 600 psi concrete when hard grade steel is used as against 0.64% steel for 600 psi concrete when intermediate grade steel is used.

As mentioned previously, the amount of steel needed depends on the tensile strength of the concrete; therefore, within reason, the lower the strength of concrete, the lower the percentage of steel required. Investigations of all continuous pavement jobs built in Texas show that the steel cost is approximately one-fourth the concrete cost. By averaging all completed jobs, the cost of concrete in place was $15.10 per cu yd and the cost of steel in place was $0.09 per lb. Using these figures, and considering a 24-ft wide pavement, the total cost of the concrete and steel per lineal foot of roadway for various strength concretes and design loads was computed for comparison purposes.

Fig. 6 shows the comparative design thicknesses, steel ratios, and pavement costs for three design load conditions and three different concrete design strengths using intermediate grade steel. The major fact emerging from Fig. 6 is that a design calling for a seven day flexural strength of 650 psi concrete is more expensive than a design calling for a seven day flexural strength of 600 psi concrete.

Whereas the actual costs obtained are of little value because prices are changing so rapidly, the trend presented is of value. Calling for a seven day modulus of rupture of 600 psi instead of 650 psi could yield a saving of 4% according to the figures herein used and, in all probability, the savings would even be greater due to the fact that by calling for a 650 psi concrete, the contractor is forced to initially mix approximately an 800 psi concrete and work down to insure compliance with the specification. This, of course, raises the cost of concrete. Obviously, by lowering the strength requirement the unit cost will also be lowered, thereby yielding even greater savings than those shown.

One other important point needs to be mentioned here. If a section of pavement is placed with concrete considerably stronger than the design strength, the forces transmitted to the steel from restrained pavement volume changes will cause failure by yielding of the steel at the cracks in the concrete, and these cracks open progressively wider as the steel can no longer hold them closed. In turn, these excessive crack widths create an edge loading condition, and the loads eventually cause a pavement failure at these locations.

Comparisons of Theoretical Crack Spacing Varying Bond Area and Concrete Strength.—Using the conditions assumed for the conventional design

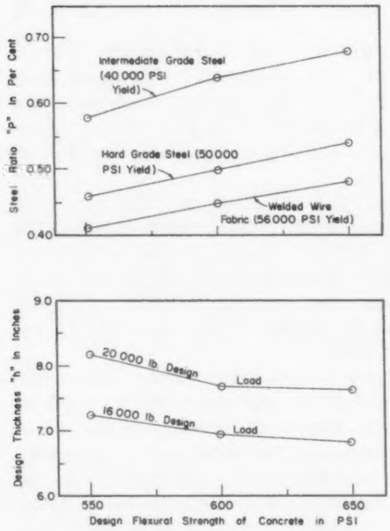

FIG. 5.—DESIGN COMPARISONS USING DIFFERENT TYPES OF STEEL AND DESIGN LOADS FOR VARIOUS FLEXURAL STRENGTHS

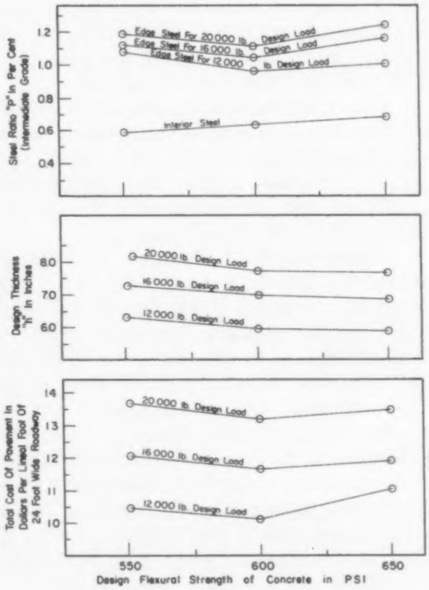

FIG. 6.—COMPARATIVE STEEL RATIOS, DESIGN THICKNESSES, AND TOTAL COST OF PAVEMENT PER LINEAL FOOT OF 24 FT ROADWAY FOR VARIOUS DESIGN LOADS AND DESIGN CONCRETE FLEXURAL STRENGTHS

(650 psi concrete), and assuming the use of No. 4 bars for 0.68% longitudinal steel reinforcement, the theoretical crack spacing due to forces developed from restrained volume changes is 23.5 in. By increasing our bar size to No. 6 and holding all other factors constant, the theoretical crack spacing is 34.4 in. This points out the major effect of bond area on the eventual crack spacing. It is readily seen that for a given length of pavement the sum total of crack widths must be the same for every size of reinforcing steel bars if everything else is held constant; therefore, the smaller the size of bar (that gives the greater bond area) the more cracks and, consequently, the smaller the crack width.

This same discussion can be used for various concrete strengths neglecting changes in the shrinkage coefficient. If concrete strength is increased to 750 psi (seven day flexural design strength), using No. 4 bars, the theoretical crack spacing is increased to 31.2 in., that is over a 32% increase in crack spacing. Consequently, crack widths will be over 32% wider for 750 psi concrete than for 650 psi concrete.

All of these theoretical facts tend to add evidence to the earlier contention that lower strengths of concrete are desirable and higher strengths of concrete will have fewer, but considerably wider cracks creating the very condition continuous pavement is designed to eliminate.

PAST EXPERIENCE

General.—For the purposes of supporting the proposed design criteria, the data gathered from continuously reinforced concrete pavements constructed in this country will be discussed from the aspect of crack spacing. Investigations of this past experience revealed additional considerations necessary to the proper design of a continuous pavement. The location of the longitudinal steel in relation to the vertical depth of the slab is also worthy of discussion.

Crack Spacing.—The design of every continuously reinforced concrete pavement has been based on the criteria that the function of the steel is to control the cracking. This control offered by the steel is not for the prevention of cracking, although this would be an ultimate condition, but rather for the purpose of keeping the crack at an optimum width. The optimum width should be small enough to (1) prevent the entrance of water and (2) provide adequate load transfer through aggregate interlock. If these conditions are satisfied, then the crack would have no detrimental effect on the ultimate life of the pavement.

Because studies made on projects constructed in the past have shown that the crack width is directly proportional to crack spacing in a linear relation (see Fig. 7), the factors influencing crack width will be examined from the aspect of crack spacing. Past experience shows that for the same concrete strengths the percent of steel, the bond area, and the setting temperature of the concrete have an effect on the crack spacing.

Percent Steel.—All designers in the past have logically assumed that the crack spacing was inversely proportional to the percent of steel. A plot of the crack spacing versus the percent steel for projects in five states shows this assumption to be correct (see Fig. 8). An examination of this graph shows that an increase in the percent steel beyond 1% does not materially affect the crack spacing, whereas a small reduction below 0.4% has a tremendous effect. Caution against reducing the steel below 0.4% cannot be overemphasized, for under certain conditions crack spacings larger than 16 ft could produce detrimental crack widths. Although there are many other factors to consider, this graph does leave the impression that the desirable percent of steel for obtaining the most advantageous use of the steel is in the range of 0.4% - 1.0%. It might be noted here that the percentage of steel obtained by the design procedure presented in this paper falls within this range.

Bond Area.—In the past, designers have considered the bond area from the aspect of providing enough bond to develop the design stresses. No publications were found in which discussion was given to the possibility of the total bond area per volume of concrete affecting the crack spacing. A plot of crack spacing versus the bond area per volume of concrete for projects in different

states shows an inversely proportional linear relation between these two factors (see Fig. 9). This graph gives evidence that, under the same conditions, pavements with varying percentages of steel, but having equal bond area per

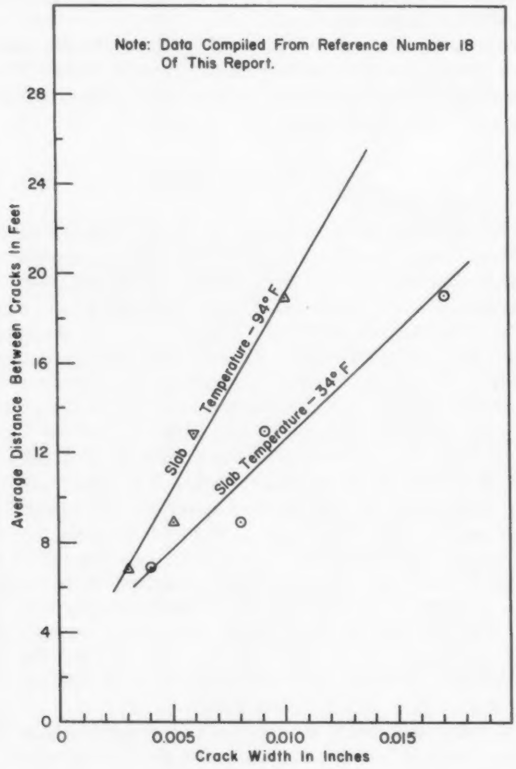

FIG. 7.—RELATIONSHIP BETWEEN CRACK WIDTH AND CRACK SPACING FOR TWO SLAB TEMPERATURES

volume of concrete would have equal crack spacings. For the purpose of correlating the diameter and the bond area of the steel, the formula for percent steel and the bond area per volume of concrete are equated as follows:

$$P = \frac{A_s}{A_c} = \frac{\pi D^2}{4 h Y} \qquad (12)$$

Therefore,

$$Y = \frac{\pi D^2}{4 h P} \qquad (13)$$

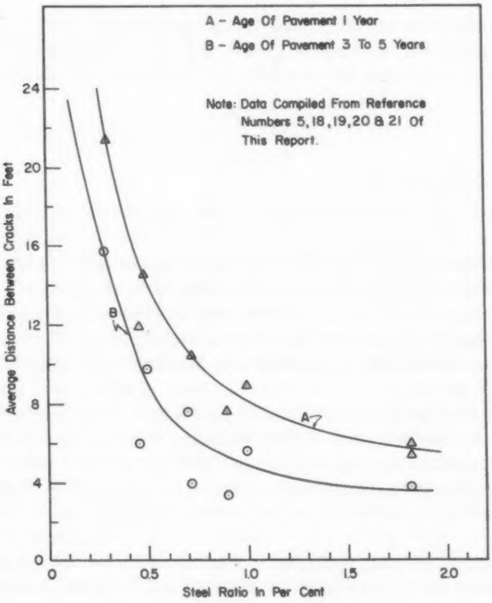

FIG. 8.—RELATIONSHIP BETWEEN STEEL RATIO AND CRACK SPACING

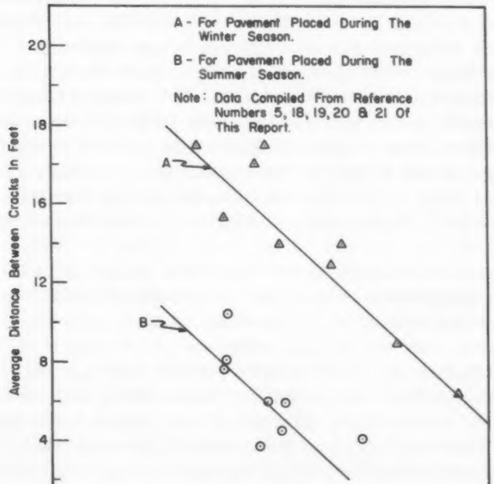

FIG. 9.—RELATIONSHIP BETWEEN BOND AREA PER CONCRETE VOLUME AND CRACK SPACING

Also

$$Q = \frac{A_b}{V_c} = \frac{1\ \pi\ D}{1\ h\ Y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Therefore

$$Y = \frac{\pi\ D}{h\ Q} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

By equating Eqs. 13 and 15

$$Q = \frac{4\ P}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

in which Y is the spacing of longitudinal reinforcing bars in inches, A_b denotes the total bond area of longitudinal reinforcing bars in square inches, V_c is the total volume of concrete in cubic inches, and Q refers to the ratio of bond area to concrete volume in square inches per cubic inch.

To have an optimum crack spacing the magnitude of the ratio Q should not be less than 0.03 sq in. per cu in. for summer placing, and 0.04 sq in. per cu in. for late fall and early spring placing. Inasmuch as the goal of the design is to obtain an optimum crack width, sufficient bond area should be provided to cause the formation of a new crack for stress relief in the concrete rather than further contraction at the existing cracks. This factor should be investigated thoroughly before arriving at bar sizes and spacings for a given percentage of steel.

Atmospheric Curing Temperature.—The effect of the atmospheric temperature during curing is vividly pointed out in Fig. 9. Line A represents the Q-crack spacing relation for projects placed during the late fall and Line B for projects placed in the summer. The figure shows that a continuously reinforced pavement placed in the summer season will have a much closer average crack spacing than a similar pavement placed in the winter season. This relation points out the need for considering the time of year a pavement is to be constructed when designing for an optimum crack spacing.

By reverting back to the theory for temperature stresses, the explanation for this observation becomes self-evident. The greater temperature drop for the projects placed in the summer results in larger stresses, hence, more cracking and closer crack spacing than those placed in the cooler weather.

From this discussion it may be concluded that, to obtain an optimum crack spacing, the bond area of the steel and the season of the year the concrete is to be placed must both be taken into consideration when determining the percent steel to use.

Steel Placement.—Although there has been some debate on the vertical placement of the longitudinal steel, the concensus of opinion among engineers was to place it at the mid-depth of the slab. Recent experiments in which the vertical placement was varied for different percentages of steel have confirmed this assumption (4). The results for the steel placed at the mid-depth showed less total vertical movement for wheel loads and less steel stress at the cracks due to temperature differential and wheel loads than other placement positions. The steel being at mid-depth allows the slab to give equal resistance to downward deflection under the load and upward deflection at points some distance away. This placement also helps resist upward soil pressures created by the "potential vertical rise" (16) of the soil.

In addition, a further supporting factor would be the fact that the combination of eccentric reinforcement and shrinkage cause warping. Tests show symmetrical sections do not warp regardless of the magnitude of concrete shrinkage (17).

SUMMARY

Discussion of the Texas Design.—The Highway Design Division of the Texas Highway Department is tentatively (as of 1960) recommending a design calling for an 8 in. concrete slab continuously (longitudinally) reinforced with 0.5% intermediate grade steel for up to 20,000 lb design wheel loads. It is also of interest to note that recommended typical specifications for this design using average aggregates found in Texas place a maximum allowable seven-day concrete flexural strength of 675 psi, as well as a minimum flexural strength (550 psi).

All pavements built to this design (as of 1960) are performing very well. According to the design theories presented herein, the Texas design has insufficient steel, since only intermediate grade is required, to adequately carry the internal forces, however, as mentioned previously, this design approach has been developed on the conservative side and has not considered the important factors of subgrade friction, partial slab movement, and crack pattern development while the concrete is still "green" or weak. Undoubtedly, the magnitude of the internal forces is reduced by each of the preceding factors, that would allow the use of lower percentage of steel than is theoretically required. Texas believes that 0.5% steel will be sufficient so long as sufficient bond area is maintained. If the designer should desire to reduce the theoretical amounts of steel required, sound engineering judgment alone must be used until more information is obtained on the previously noted factors.

Conclusions.—1. A simple and reasonable approach has been established to a very difficult and complex problem of continuously reinforced concrete pavement design that will yield economical designs for a variety of conditions and will serve as a basis for experimental investigations aimed at arriving at a more thorough understanding of continuously reinforced concrete pavement.

2. Hard grade steel or welded wire fabric should be used for longitudinal steel in continuously reinforced concrete pavement.

3. Concrete with a strength lower than that heretofore considered to be the minimum necessary should be used for continuous pavement in order that maximum utilization of the reinforcing steel may be realized.

4. Current specifications should be amended to provide a maximum as well as a minimum seven day modulus of rupture with values depending on local conditions.

5. The bond area of the steel and the season of the year the concrete is to be placed should be taken into consideration when determining the percent of steel.

6. The longitudinal steel should be placed at the mid-depth of slab.

7. A pavement design for concrete having a seven-day flexural strength range of between 575 psi minimum and 675 psi maximum (as determined by center point loading), calling for a thickness of 7.0 in. (rounded up from a 6.8

in. design thickness), with a longitudinal interior steel ratio of 0.5%, (using hard grade steel) will be sufficient to withstand the greatest current loads.

APPENDIX I.—WESTERGAARD'S DESIGN CHARTS

The following design charts (Figs. 10 and 11) may be used for computing the required interior thicknesses, using Eq. 1 and required edge thickness, using Eq. 11, for values of E varying from 1 million psi to 4 million psi.

APPENDIX. II.—BIBLIOGRAPHY

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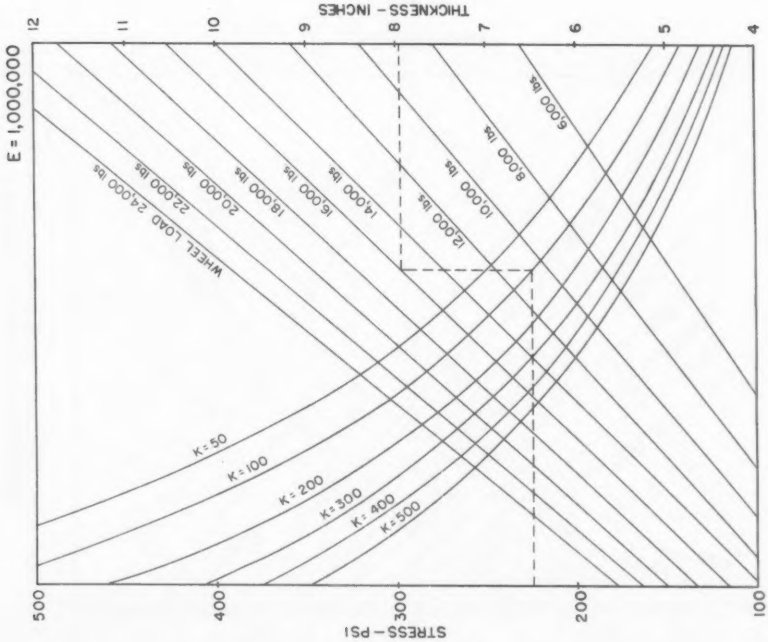

FIG. 10(a).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S INTERIOR LOADING (THEORETICAL)

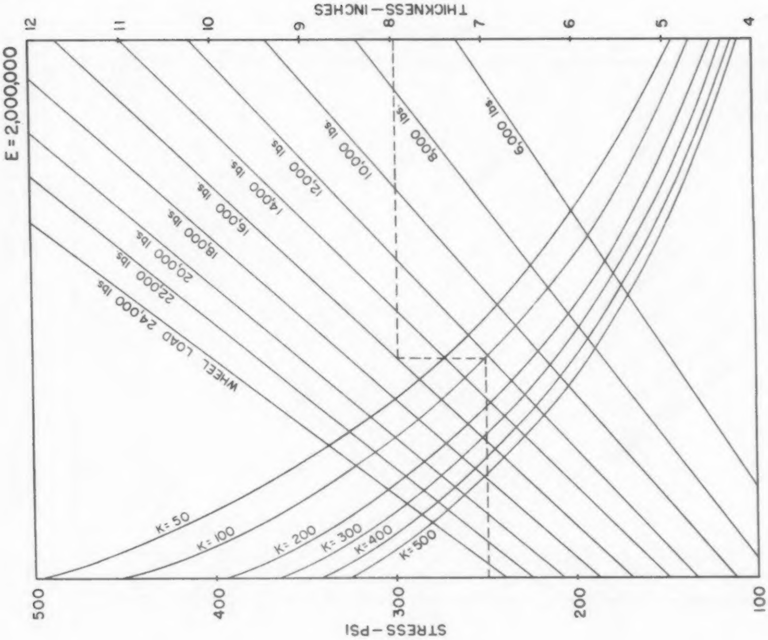

FIG. 10(b).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S INTERIOR LOADING (THEORETICAL)

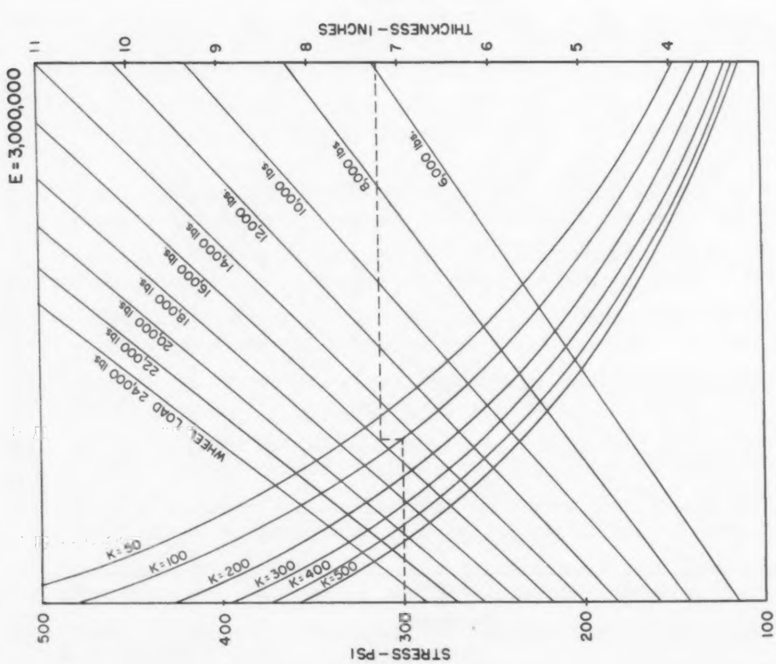

FIG. 10(c).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S INTERIOR LOADING (THEORETICAL)

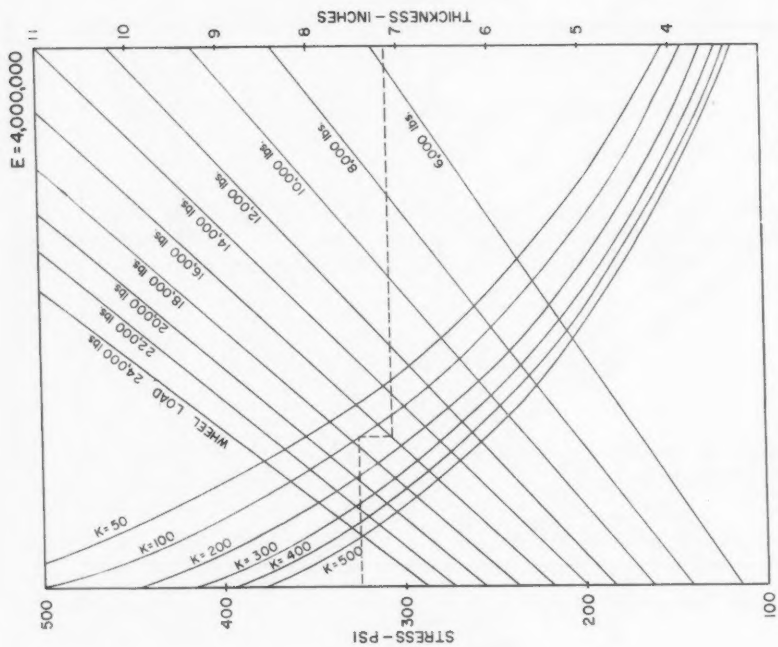

FIG. 10(d).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S INTERIOR LOADING (THEORETICAL)

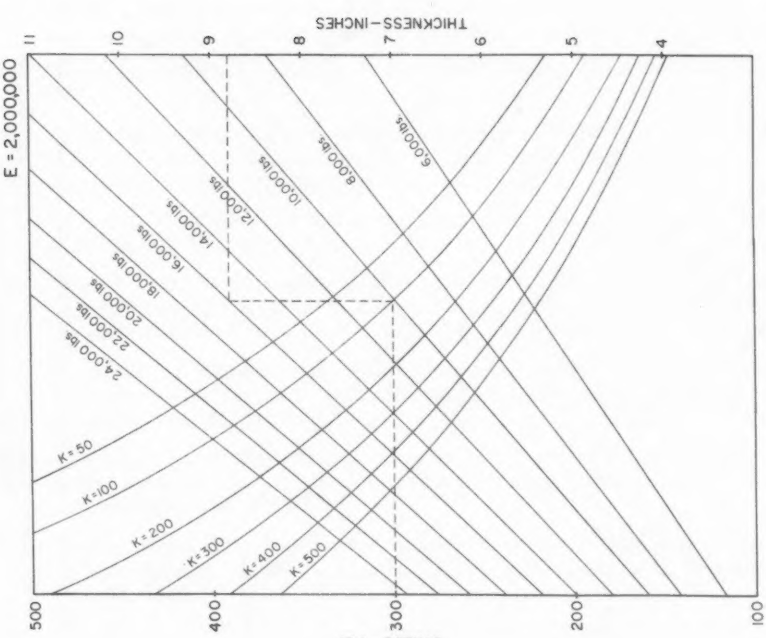

FIG. 11(b).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S EDGE LOADING

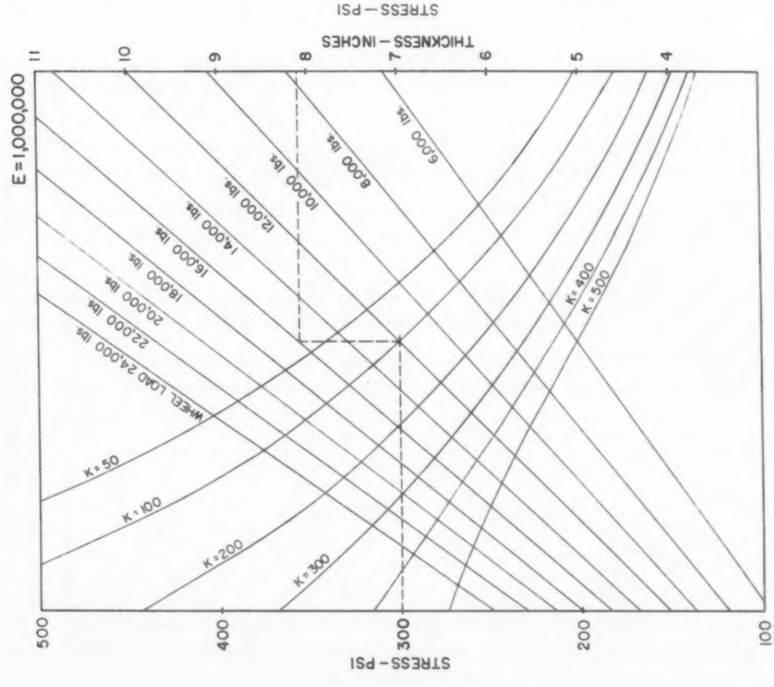

FIG. 11(a).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S EDGE LOADING

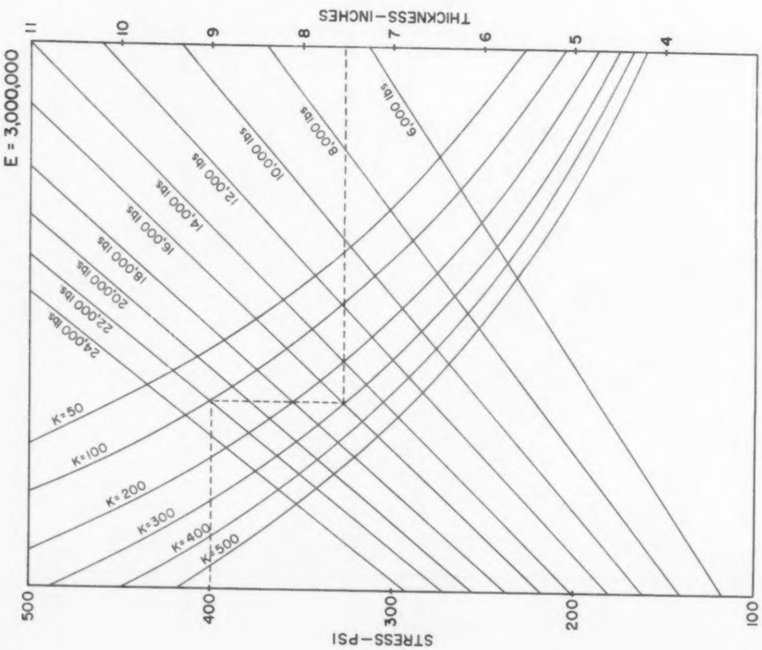

FIG. 11(c).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S EDGE LOADING

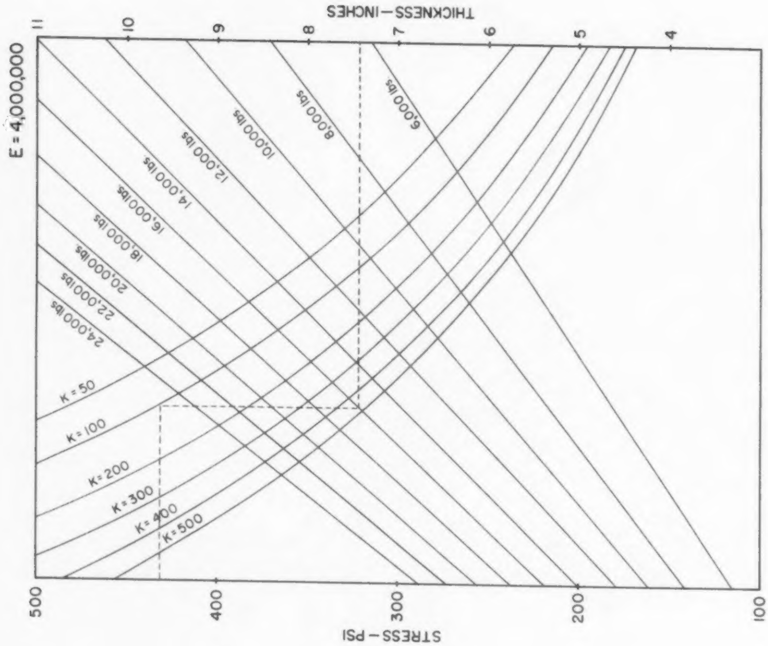

FIG. 11(d).—DESIGN CHART FOR CONCRETE PAVEMENT WESTERGAARD'S EDGE LOADING

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APPENDIX III.—NOTATION

The algebraic symbols used in this paper are defined as follows:

S_i = Maximum allowable interior modulus of rupture or flexural stress (PSI) in the pavement due to load;

S_e = Maximum allowable edge modulus of rupture or flexural stress (PSI) in the pavement due to load;

W = Design wheel load in pounds;

d = Trial thickness of pavement in inches;

h = Actual design thickness of pavement in inches;

k = Modulus of subgrade reaction in pounds per cubic inch;

a = (1) Radius of a small circle over the area of which W is assumed to be distributed uniformly when the load is at a considerable distance from the edge; or (2) radius of a small semi-circle over the area of which W is assumed to be distributed uniformly when the load is at the edge, the center of the semi-circle being at the edge;

A = Area; A_s = Area Steel; A_c = Area Concrete;

P = Steel area ratio, A_s/A_c;

S'_c = Tensile strength of the concrete in PSI;

S_s = Yield strength of the steel in PSI;

μ = Poisson's ratio;

z = Shrinkage coefficient of concrete;

ϵ = Thermal coefficient of concrete and steel;

T = Total drop in temperature in ° F;

E = Modulus of elasticity; E_s = Steel; E_c = Concrete;

n = Ratio of modulus of elasticities of steel to concrete $(n = E_s/E_c)$;

u = Unit bond stress between the concrete and steel in PSI;

Σ_o = Sum of perimeters of bars in inches;

q = Ratio of perimeter of bars to area $(q = \Sigma_o/A_s;\ P \cdot q = \Sigma_o/A_c)$;

X = Distance between cracks in inches;

A'_c = Equivalent area of concrete $(A'_c = n\ A_s)$;

P' = Additional steel ratio needed along the longitudinal edges of the pavement;

Y = Spacing of longitudinal reinforcing bars in inches;

A_b = Total bond area of longitudinal reinforcing bars in square inches;

V_c = Total volume of concrete in cubic inches;

D = Diameter of reinforcing bars in inches; and

Q = Ratio of bond area to concrete volume in in^2/in^3 $(Q = A_b/V_c)$.

Journal of the

HIGHWAY DIVISION

Proceedings of the American Society of Civil Engineers

DISCUSSION

ELECTRONIC COMPUTER IN HIGHWAY ENGINEERING[a]

Closure by L. R. Schureman

L. R. SCHUREMAN,[1] M. ASCE.—The writer appreciates Mr. Carlson's interest in discussing his paper and agrees with him that careful planning is essential to the development of effective computer programs.

The use of electronic computers in highway engineering and administration is relatively new and while progress has been notable there is much yet to be developed in obtaining maximum advantage from these devices, particularly in the field of programming. For any given problem, the solution is the responsibility of the programmer. The computer will simply perform the computations in accordance with the detailed plan of operation he prepares.

The example given in Mr. Carlson's discussion illustrates very well the way in which programs may be combined to operate in series. This has also proved effective in highway design where several component programs are combined and arranged so that the output from one serves as input for another with provision for review and decision by the engineers as necessary during the overall process.

With the larger internal storage capacities becoming available in computers, correspondingly greater opportunity is afforded for increased flexibility and for the complete solution of an engineering problem in a single computer run, as opposed to a series of programs used in sequence. It is essential however, regardless of the type of problem involved, to plan and develop the program so that the computer adequately serves the engineer, providing him with the information he needs for sound engineering decisions.

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[a] September, 1959, by L. R. Schrueman.

[1] Highway Engr., U. S. Bureau of Public Roads, Washington, D. C.

PASSENGER DATA FOR URBAN TRANSPORTATION PLANNING[a]

Closure by Nathan Cherniack

NATHAN CHERNIACK,[1] F. ASCE.—The writer is grateful to all of the discussers for contributing their thinking to a subject that, today, is highly controversial. In his closure the writer will concentrate in presenting responses to five of the discussers who differed most violently from the author's conclusion that "Today, capital investments in new systems of rail mass transit facilities do not appear justified."

Response to Mr. Rannell's Discussion.— Mr. Rannells puts this question: "The question whether, in 'the grand accounting,' movement across the Hudson might be better served by something different - for example, a combined rail-highway program - is avoided in this paper." In answer to this question the author would refer Mr. Rannells back to Table 4, which was presented to show that between New Jersey and New York, there is now in existence a combined rail-highway passenger system across the Hudson River that faithfully reflects present day choices of railroads, busses or autos, as modes of travel, by the different segments of trans-Hudson passengers, in their day-to-day trans-Hudson journeys.

Mr. Rannells makes this pertinent comment: "If New York City is to continue viable, means must be found for more balanced utilization of mass transport facilities by selective development of the more useful and potentially economical segments of the passenger rail network."

The writer agrees with this statement. Where there are already in being, rail rapid transit facilities that serve the Central Business District and they represent considerable sunk capital, let us by all means rehabilitate, improve, expand, and make them so attractive that they not only do not continue to be under-utilized, but that they be utilized as intensively as is practicable.

Response to Mr. Noble's Discussion.—Because Mr. Noble was practically in agreement with the author's points, supported by his own wealth of experience in highway, traffic, and transportation engineering, he asked Mr. Charles J. Lietwiler, whom he describes as a "transit enthusiast," to comment also on the author's paper. One of Mr. Lietwiler's comments is:

> "Must we continue to build these low capacity (in terms of the people) high-cost facilities (that is, expressways) simply because they are in use 24 hours a day?"

An expressway can handle 720 buses an hour past any given point, and each bus can carry 50 seated passengers. The expressway can thus handle 36,000 passengers an hour, past any given point. This is not low capacity by any standard, either, in vehicles handled or in terms of people carried.

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[a] December, 1959, by Nathan Cherniak.

[1] Economist, The Port of New York Authority.

Expressways are in use 24 hr a day, while railroad tracks are in use some 4 to 6 hr a day. This, at least, is one effective measure of their comparative economies.

Mr. Lietwiler then proceeds: "A railroad track on which both freight and passenger service are being operated is certainly in use more than '2 or 3 morning hours' and '2 or 3 afternoon hours'." This is true, however, the Pennsylvania Railroad carries no freight into its Manhattan Pennsylvania Station, nor does the New York Central Railroad carry freight into its Grand Central Terminal on Manhattan.

Mr. Lietwiler then comes to rest on a well worn cliche as follows: "This attitude fails to recognize the basic transport problem, which is the movement of people, not the movement of automobiles, busses or trains." The author recognizes this basic transport problem and suggests that express buses will move people en masse, from suburbs into a central business district, on trunk line expressways in volumes of at least 25,000 persons per hour, as evidenced by the rush hour bus passenger volumes that move through the Lincoln Tunnel, and through The Port of New York Authority's midtown Manhattan bus terminal.

Response to Mr. Kashin's Discussion.—Mr. Kashin has raised questions regarding some statements that the author has made. He states that, "The author is assuming that an expressway lane is used to capacity at all times and that the mere movement of vehicles is efficient."

The author has pointed out that a single expressway lane can handle at least 25,000 bus passengers in rush hours on weekdays. The same expressway lane could and does also handle autos and trucks as well as buses on Sundays and holidays when bus traffic falls off. On the other hand, a railroad track, even where it is used to capacity (in rush hours on weekdays) usually remains practically in disuse on Sundays and holidays throughout the year. An expressway lane is thus used more intensively throughout the year, for the movement of both vehicles and people, than a railroad track. To that extent an expressway lane is more economical. It functions both as a passenger and a vehicular facility.

Mr. Kashin continues: "He (the author) assumes that suburban railroad tracks are used exclusively for suburban trains, ignoring the freight and intercity passenger trains which also share these same tracks." The four railroad tracks into the Grand Central Terminal of the New York Central Railroad handle passenger trains exclusively. They do not handle any freight trains.

Further on Mr. Kashin points out that, "In the rush hour, Grand Central Terminal and Pennsylvania Station are each handling between 45,000 and 55,000 passengers." The Grand Central Terminal makes use of three of the four tracks into the terminal in the morning rush hours. It thus handles train passengers in peak periods, when it is up to capacity, at a rate of about 17,000 passengers an hour per track. The Port Authority Bus Terminal, on the other hand, handles 20,000 - 25,000 passengers in the rush hour in buses via only one equivalent lane, either in the Lincoln Tunnel or on the approach expressway. In fact, one tunnel lane or expressway lane could handle more than this demonstrated rush hour passenger volume and the two ramp lanes into and the two out of the bus terminal, could handle considerably more, as they probably will after the expansion of the terminal.

Response to Discussion by Mr. Hawley Simpson.— Mr. Simpson cites the Congress Street Expressway in Chicago and quotes figures to ascertain that: "Even on a 24-hour basis Congress Street Rapid Transit, with one track in each direction, is carrying half as many passengers as the expressway in four lanes."

Let us look at the record of the Congress Street Expressway from another vantage point. The entire right-of-way of the Congress Street limited access route devotes four expressway lanes in each direction for free vehicle traffic, and two lanes in each direction are set aside for rapid transit. There are thus twice as many vehicle expressway lanes as rail rapid transit lanes. The expressway vehicle lanes cost $2\frac{1}{2}$ times the uncompleted rapid transit lanes. (Only one track has been constructed in each direction.)

To be sure, in the peak hour, the expressway lanes carry only 68% as much as the passengers handled by the transit lanes, but over the 24 hr on weekdays, day in and day out, the expressway lanes have been carrying about twice as many people as the transit lanes. Moreover, on about 115 Saturdays, Sundays and holidays, or for about 30% of the year, the expressway lanes will probably handle about five times as many people as the transit lanes, because on weekends and holidays the transit lanes probably will not handle half the weekday passengers, while the expressway lanes will carry more cars and they will be more heavily loaded.

A careful analysis of present usage, by people, of the Congress Street Expressway and rapid transit lanes, over the entire year, would probably reveal (a) that it is the expressway lanes that are meeting most the off-hour weekday travel, "reverse travel," and the week-end leisure-time travel demand, and (b) that the expressway lanes are therefore not only economically justified, but constitute an absolutely essential complementary element in a balanced system of transportation supply, to meet the broad spectrum of Chicago's passenger transportation demand from the westerly direction.

Mr. Simpson says that "Buses can deliver to the 34th - 42nd Street area, but a transfer is required to most central destinations because the Port Authority (bus) terminal is near the western edge of Manhattan, three-quarters of a mile from Fifth Avenue." And a little further on, contrasting railway and highway travel, he says, "---railways find very substantial acceptance provided reasonably good central district delivery can be accomplished. That type of central delivery is well accomplished at the Grand Central and Pennsylvania Stations."

From the Pennsylvania Station (PRR and LIRR), a transfer is also required to most central destinations because it, like the Port Authority Bus Terminal, is also near the western edge of Manhattan, three-quarters of a mile from Fifth Avenue. Again, from the Grand Central Station (New York Central and New Haven Railroads), a transfer is also required to most central destinations because it is near the eastern edge of Manhattan, more than three-quarters of a mile from Times Square.

Mr. Simpson switches from a comparison of suburban express buses with suburban rail transit, to a comparison with inner core and central business district (CBD) rapid transit systems, like those of the New York subways. He says: "The capacity achieved in this bus terminal is accomplished under unique conditions, without multiple-point delivery within the core of Manhattan. Were the buses to operate in station-to-station service with stops as frequent as on the New York subways, no such capacity could be achieved." This point is granted; however, to provide a multiple-point distributing system within a CBD is definitely not the function of a suburban bus system. Its function is to link suburbs to a strategic location within a CBD. If the CBD is large, a local circulating common carrier system must be provided to distribute suburban passengers within the CBD, whether they arrive at a railroad station or at a bus terminal.

Of course, it is physically impossible for any of the railroads to deliver their passengers to their ultimate CBD destinations. Their passengers do have to transfer to local common carriers. On the other hand, Mr. Simpson apparently makes the implicit assumption that because a suburban bus can travel on the streets of the CBD, that it must necessarily deliver its passengers to the ultimate destinations. Not at all! Suburban buses need only deliver their passengers to a bus terminal. Providing a convenient, fast-circulating common carrier system within a large CBD is one of several major problems, in providing adequate overall urban mass transportation for suburbanites.

Mr. Simpson further elaborates on the comparison between a suburban bus terminal and a New York subway station by comparing the space required to deliver passengers, of one with the other. In the first place, Mr. Simpson compares the capacities of a stub end suburban bus terminal with that of a New York City subway through station. These are two radically different types of mass transit passenger stations.

In the second place, the 450 buses in the peak hour were handled only on the suburban or short haul level (see Fig. 8), where the 72 loading berths are located. The other 40 berthing spaces are on the long haul bus level. Moreover, most of the 20,000-plus bus passenger arrivals, in the morning rush hour, in 450 buses, were unloaded on an unloading platform, along the south side of the bus terminal, with some 14 bus berths, and not at the 112 berthing spaces, as Mr. Simpson stated. At 2 min unloading time per bus, or at 30 bus stops per hour, the 14 bus berths could handle more than 400 buses an hour. To be sure, 72 loading berths are necessary to handle the more than 400 bus departures in the rush hour (as of 1960).

Now let us examine the other side of the coin, "A rapid transit line operating 10-car trains on 90 sec headways, at 100 passengers per car, requires only 10-car spaces for twice as many passengers." The 40,000 train passengers that arrive at the station in the rush hour are usually not all unloaded at the 10-car spaces. Only a fraction of them are usually unloaded, whether at a through station or at a terminal station. Consequently, it would seem that Mr. Simpson's previous comparison between the Port Authority's bus terminal and a rapid transit line would appear to be somewhat unfair.

To show how unfair the previous comparison really is, a similar unfair comparison could be made between a specific stub end railroad passenger terminal and a closely related New York City Subway station.

The Grand Central Terminal, a stub end railroad terminal, requires 3 inbound tracks, and a total of 48 platforms of 10-car spaces each to handle 48,000 persons, in the morning rush hours. On the other hand, through the Grand Central Station, a through station on the New York City's Lexington Avenue subway in the Grand Central Terminal area, 45,000 persons can move in the rush hour (30 ten-car trains per hour at 150 persons per car) in one direction with a stop at its platform of 10-car spaces, but not all 45,000 passengers would get on and off at that station, within the 90 sec headway.

For a more correct comparison of comparables, between the Grand Central Terminal and rapid transit station performance, the author obtained these figures from the New York City Transit Authority. At the Grand Central subway station complex, which passengers use to depart from, (a) via the Lexington Avenue subway to the north and south, (b) via the 42nd Street shuttle subway to the west toward Times Square, and (c) to the east via the Flushing subway to Queens, a total of about 40,000 passengers (actually 39,131 on Wednesday, March 16, 1960, between 5 and 6 p.m.) paid fares in an afternoon rush hour.

This subway complex, however, requires more than just a 10-car space platform. There are actually 8 platform "edges" with a total of 55-car spaces to handle the 40,000 passengers who get on and off, at this subway station complex.

Mr. Simpson appears to indulge in another unfair comparison when he says: "A stub end bus terminal such as the Port Authority terminal requires more berthing time than would a through station, and thus more berths for the same number of buses. To relieve the situation, the St. Louis proposal includes an elevated downtown bus loop having a total length of 22 blocks with 80 berths, - designed to handle 548 buses per hour. The average headway per berth works out at about 2 minutes, 12 seconds." The continuous, elevated bus platform proposed for Downtown St. Louis by Gilman and Company is, in the opinion of this writer, a practical and effective proposal for bus passenger distribution within that CBD. Under this proposal, suburban buses themselves could actually circulate in the CBD without undue delay, and thus effectuate multiple-point delivery of passengers to within walking distances of their ultimate CBD destinations. And yet Mr. Simpson attacks this proposal with the same type of unfair comparison between a suburban bus terminal and an inner core rapid transit subway station for multiple-point CBD distribution, when he says: "This plan requires 20 berths for half as many passengers as could be handled in a 10-car station in rail rapid transit, a station efficiency factor of 25%."

Response to Mr. Harold M. Lewis' Discussion.—Mr. Lewis takes the author to task when he says that, "Ten years is a very short time in the history of passenger transportation in the New York-New Jersey Metropolitan Area." He goes on to explain that future trans-Hudson movement between New York City and New Jersey would benefit from an examination of some of the conclusions and predictions reached by earlier studies.

Mr. Lewis then cites a study by the Regional Plan Association[2] as the kind of study he had in mind. Mr. Lewis went on to quote some of the conclusions.

As the Late Governor Al Smith of New York was wont to say, "Let's look at the record." Let us look at the RPA Information Bulletin No. 25, dated June 17, 1935, but a little more closely and in a bit more detail than Mr. Lewis did.

The introduction of Bulletin No. 25 states: "The Port of New York Authority has cooperated by making available all figures in their files, by the taking of traffic counts and by the compilation and analysis of material." The writer, being one of the Port Authority's representatives in the cooperative project, feels qualified to refer to Bulletin No. 25 with some degree of authority.

When Mr. Lewis therefore complains that "ten years (1948-1958) is a very short time in the history of passenger transportation in the New York-New Jersey Metropolitan Area," the author must call Mr. Lewis' attention to the fact that Bulletin No. 25, too, analyzed "trans-Hudson movement of all kinds for the decade 1924-1934." "It analyzed trends of the New Jersey railroads 1911-1934," but the author also analyzed overall trends of trans-Hudson movements by all modes (autos, buses, ferry pedestrians and railroads) for the 48 years, 1911-1959. The railroad passenger data were the same as in Bulletin No. 25. As Mr. Lewis indicates, "This was based on the same figures as are shown in that portion of Fig. 1 in Mr. Cherniack's paper up to the year 1934..."

It takes a wealth of data to examine, in some detail, the passenger movement experiences of two "benchmark" years, a decade apart whether the decade

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[2] "Survey Throws Light on Need for New Facilities Across the Hudson River," Information Bulletin No. 25, Regional Plan Association, Inc., June 17, 1935.

is 1924-1934 or 1948 and 1958. The writer can vouch for the extreme difficulties in assembling the data for both ten year periods.

Now let us look a little more closely and in a little more detail at the "predictions reached by earlier studies and seeing to what extent these have proved valid." Again let us refer to Bulletin No. 25 that Mr. Lewis quotes. The writer wishes now to quote other sections of Bulletin No. 25 and compare them with developments in the 25 yr that have elapsed since then.

Bulletin No. 25 contains several paragraphs pertinent to the current period. It is stated that there is a decrease in rail service as the number of autos and bus lines increase, and thus there was a decrease in trans-Hudson railroad passenger service for this period. However, it is stated, that, "a measurement of the actual absorption of rail traffic by motor vehicles is beset with many difficulties. The problem of accounting for rail decline involves three factors: diversion to motor vehicles, temporary depression losses, and permanent reduction because of decentralization."

The decline in rail passengers was from 76.2% of the total in 1925, to 61.2% of the total in 1934 to 1959 when the percentage of the total trans-Hudson passengers carried by the railroads has declined to 20%.

Bulletin No. 25 is prophetic of the current period in stating that: "As a means of inter-urban transportation in the New York Metropolitan Area, it is probably true that the bus has come to stay permanently." Following a listing of the advantages of bus traffic it was predicted that "diversions to buses may be exceedingly difficult for railroads to recover unless they inaugurate new types of services, such as the very frequent service possible with some form of electrical operation." It is also predicted that the use of the private auto will increase despite its shortcomings. However, the increase in the use of private autos is not seen as being permanent based on the idea that as more river crossings are needed, more people will return to rail commuting. "Diversions to private automobiles, therefore, may be regarded as partly temporary and partly permanent railroad passenger losses."

Bulletin No. 25, describes the process of diversion of trans-Hudson passengers from rails to buses that began in earnest some 25 yr ago (during the 1930's). It is pointed out that in conjunction with the convenience of the George Washington Bridge and its link with the New York subway system the rise in rail fares made it attractive for many residents of Bergan County, New Jersey to switch from rail communtation to the bus and subway combination.

In conjunction with the construction of the lower deck of the George Washington Bridge, The Port of New York Authority will construct a new terminal for suburban buses between West 178th and West 179th Streets, running east and west, and between Fort Washington Avenue (running north and south) at the Bridge Plaza and St. Nicholas Avenue two blocks to the east, to accommodate the bus and bus passenger movements that have developed from Bergen County, N. J. and Rockland County, N. Y. to the north. These bus passenger movements will continue to expand across the George Washington Bridge after the lower deck is completed.

Bulletin No. 25 states that increases in Lackawanna (DL&WRR) fares, following the electrification of this line, coupled with the decrease in passenger car tolls for the trans-Hudson ferries, made it attractive for residents of Essex County, N. J. to drive into Manhattan.

In addition the partial or intermittent work week drove many New Jersey commuters to abandon the railroad in favor of one of several possible com-

binations of travel equally convenient, and in many instances more economical, than commutation rail travel. The modes of travel enumerated were; (1) using the bus from home to business in Manhattan, (2) using the bus from home to Journal Square (Jersey City, N. J.) and then entered Manhattan via the Hudson and Manhattan Railroad, (3) using own private car (in many instances clubbing together with neighbors and thus further reducing the cost), from home to Manhattan, and (4) using own car from home to some parking place in New Jersey, like Journal Square or the George Washington Bridge Plaza, and then entering Manhattan via the Hudson and Manhattan Railroad from Journal Square or via bus across the George Washington Bridge and subway to place of business.

The bulletin states that, "the net effect of all these events was to drive large numbers of commuters, as well as other passengers, from the rails temporarily, if not permanently."

After going through a detailed analysis of shifts in trans-Hudson passenger movements, Bulletin No. 25 then raised certain questions that could not definitely be answered in 1935, such as, "are the people of New Jersey tending to be less dependent on New York for business, commerce, employment and amusements? To what extent has railroad traffic been affected by this decentralization and to what extent will it be affected in the future? What place should the motor bus and private passenger vehicles have in the future system of trans-Hudson facilities? Will the midtown Hudson Tunnel (now called the Lincoln Tunnel), a vehicular tube now under construction, cause additional diversions to buses as a result of its accessibility to midtown Manhattan?" The writer, in his paper has attempted to give answers to these questions, as of 1960.

Mr. Lewis suggests that the scheduling of reverse travel may, "favor the railroads as it makes possible a payload in each direction, particularly if the New Jersey trains could run through to the Borough of Queens and pick up Long Island traffic on their return trip, and Long Island trains could run through to the edges of the Hackensack Meadows to pick up New Jersey traffic on their return trip." To achieve Mr. Lewis' suggestion, at least until 1954, all that was necessary was for the president of the Pennsylvania Railroad to decide that this proposal had a great deal of merit, and proceed to effectuate it. Mr. Lewis would agree that this particular case was certainly an exception to his complaint that "It is, generally, admitted that the obstacles have been financial and political." So apparently even in the case of the iron horse, you can only lead him to water.

Mr. Lewis states that during the 3-hr morning and afternoon rush hours, there is time to work out practical train schedules for such reverse movement as described above. Apparently, the iron horse has refused to drink at the fountain of wisdom.

After discussing Mr. Lewis' Fig. 1, he concludes, "There must be merit in an interstate loop if the proposal is so persistent!" Mr. Lewis should carefully distinguish between persistent desire and economic demand. Lack of economic demand explains why the persistent desire for an interstate loop of some sort has always encountered such difficulty in being realized.

Again Mr. Lewis makes a few observations of his own which the author wishes to comment on. He says, "The writer feels that the pendulum may have swung too far in the direction of rail to rubber and that the future may see a return to a more normal ratio." We must here distinguish between cities like New York and Chicago, which today possess an elevated and subway rail rapid

transit system, and those cities like Los Angeles and San Francisco that do not have such a rail rapid transit system and have yet to build them. If those cities that have invested capital in rail rapid transit systems that have become antiquated, will proceed to rehabilitate and improve them so as to be more attractive than they are, they may develop increased patronage in the future. In cities where rail rapid transit must first be built, low residential densities in tributary areas may be one of the important key determinants that would point to rapid transit by express buses on expressways as the proper mass transit solution before sinking new capital, at once, into rail facilities. Express buses would prove to be far more economical, fully adequate for decades to come, and much more flexible.

Mr. Lewis also makes this observation, "While San Francisco is investigating the possibility of adding rails to the Golden Gate Bridge, the planned rail facilities in the original design for a lower deck on the George Washington Bridge across the Hudson River are in process of being replaced by new roadways, which will probably be crowded with buses whose passengers must transfer to rapid transit in Manhattan to reach their destination." Rail passengers, perhaps also in crowded cars on fixed rail facilities with over-designed potential capacities for the needs of the low residential densities in Bergen County, would nevertheless also have to transfer, as suburban bus passengers now do, to rapid transit facilities in Manhattan to reach their destinations in Manhattan and in the Bronx.

Suburban buses on express highways in New Jersey constitute a modern form of flexible rapid transit system that is presently meeting the journey-to-work passenger travel demand, and will continue to meet it for decades to come. It is thus obviating the necessity for wastefully sinking new capital in rail facilities that would have to radiate out in several directions, and in every direction, provide huge capacities that could not possibly be utilized by the low density residential developments in the tributary areas in Bergen County.

Conclusions.—Where does the author's study and the thoughtful commentaries of the discussers eventually lead us to?

1. Trans-Hudson passenger travel in autos, buses, and railroads, despite the fact that it represents travel from only one - the New Jersey - sector, is nevertheless typical and representative of passenger travel via a balanced system of urban transportation, via individual and common carrier transportation. Also, the decade (1948-1958) chosen by the author to examine this passenger travel in depth, is equally representative, disagreements on this point by several of the discussers to the contrary, notwithstanding.

2. No urban area of any size can exist today with transportation furnished exclusively by the private automobile. The argument that the choice before us is either autos or mass transit (impliedly by rail) is entirely specious and should be dismissed out of hand.

3. The real question is, what type of fast, common carrier transportation service is best suited to the particular urban complex under consideration, - the railroad type, the rail transit type, or express buses on expressways, or some combination?

4. The cities that now possess railroad and rail transit types of common carrier transportation that represent considerable invested capital, should strive in every way not to permit them to continue to be under-utilized. They should make every effort to utilize these existing rail facilities to a maximum; rehabilitate and improve these facilities not only from a transportation standpoint, that of moving people as rapidly as possible from home to CBD, but also

from a housekeeping standpoint, that of making the stations clean, well-lighted, safe, and attractive.

5. On the other hand, cities that today do not possess rail transit, but do have or expect to have adequate urban freeways, particularly radials, that focus on their CBD's, would be well advised to utilize these freeways to a maximum as modern, flexible, mass transit systems by express buses that could yield 20,000 passenger-an-hour capacities, adequate for decades to come, in most instances. They could even provide exclusive lanes for buses, if need be, during the journey-to-work rush hour, rather than sinking huge amounts of new capital in fixed, inflexible rail transit facilities, over-designed to yield, from each direction, 40,000 passenger-an-hour capacities that would probably never be substantially utilized because of today's and future low-density, residential developments in the existing and future suburbs.

ROADBEDS ON HIGHWAYS AND AIRPORT RUNWAYS[a]

Discussion by Jacob Feld, and W. H. Campen,
L. G. Erickson, and J. R. Smith

JACOB FELD,[3] F. ASCE.—The author's approach to the preparation of foundations for pavements is as an engineering analysis of soil as a construction material, common practice prior to the popularization of intricate and detailed laboratory investigation of what were assumed to be representative samples of the soil. Listing premises in simple terms, the subject is made clear and, generally, cannot be criticized. Of course, there are many minor points of disagreement, and many examples can be cited of actual constructions that refused to follow the simple pattern of action listed in the paper. But the chief reason for such non-agreement is the fact that even uniform soil is not as uniform as a man made material or even a natural material like timber; furthermore, soil has the habit of changing its characteristics with only slight variation in moisture or in included impurities. Metal alloys have the same habit, but once the change is frozen into the solid mass, with a few exceptions of effect from vibrations, the metal remains physically stable.

At the Highway Research Board meeting in January, 1948, the late C. A. Hogentogler, then Chairman of the Soils Division, and the writer, argued at some length on the advisability of correlation with known previous experience in the action of soils, not at the complete elimination of soil tests, but giving more weight to the results of actual experience than to the results of laboratory tests. The large expansion of soil laboratory techniques and technicians at that time outweighed reports of actual job results that were being presented to the Board. At the following meeting (December, 1948) of the Highway Research Board, the writer presented a paper on "General Engineering Approach to the Classification and Identification of Soil," in which the thesis was expressed as:

> "The purpose of a soil classification and identification system for engineering control and use of soils is to permit the extrapolation of known physical actions of some soils and thereby prophesy the action of other soils under similar exterior loadings, temperature and moisture conditions. The answers desired are physical strains and stresses. The question must include that possibility. The proper classification must therefore be, not on size, color, taste or feel, but on the basis of numerical physical characteristics."

Included in that paper are references to similar approaches in the engineering use and control of other materials.

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[a] March, 1960, by Ira B. Mullis.
[3] Cons. Engr., New York, N. Y.

The author's approach to the problem is along similar lines, and the student steeped in soil mechanics may be shocked to find that engineering structures can be designed without extensive laboratory testing.

However, this is no reason for any reduction in the fine basic studies of soil structure and action carried on in the laboratory. Much has been learned of why certain soils act as they do from scientific ivenstigations of samples. One of the very fruitful examples is the X-Ray analysis of fine grained soils. W. L. Bragg,[4] in summarizing British contributions in this field, notes his identification of four distinct patterns of oxygen-silicon-aluminum ion arrangements that he compares to purl and plain stitches in knitting. The four great families of the silicate minerals are arrays of terahedra made up of four oxygen atoms, octahedra of six aluminum atoms (like two pyramids joined base to base), and single pyramids of four silicon atoms. The patterns built up are separate units of the terahedra (dense olivines), joined corner to corner in strings (asbestos), joined by three corners to make sheets (mica) and joined by all corners to make a three dimensioned framework (feldspars). It must be admitted that no general approach to the soil problem could by observation or deduction bring this information to light. Nor can it be argued, on the other hand, that such fine scientific research by itself can be the tool for engineering design.

Only a proper use of each approach to answer the questions set for each, can result in further advance of the quest for safer and more economical engineering designs. In bringing to attention the broad practical approach, the author has contributed a worthwhile paper.

W. H. CAMPEN,[5] F. ASCE, L. G. ERICKSON,[6] AND J. R. SMITH.[7]—Although Mr. Mullis includes much more or less related data in his paper, he has two main objects in view. First, that the strength of compacted road beds be measured by some suitable method instead of being judged by density, and second, that the strength be correlated with pore space and moisture content in order to obtain permanency. The writers are in general agreement with the basic principles involved, but differ in some respects, for instance, the method of testing for strength, the importance of bond in the attainment of strength, the pore space limitation, and the requirement for the percent water-filled pore space. In addition, this discussion will indicate how density may be used during construction to control the desired properties.

The writers have reported[8] the results of an investigation of six typical fine grained soils and six soil-aggregate mixtures. Each of the samples was compacted by three methods, the standard Proctor, the modified Proctor method and a method with intermediate compactive effort. The methods deliver energies of 177, 322, and 483 ft-poundals per blow and 235, 428, 965 ft-poundals per cu in. of soil. The strength was measured by a method similar to the California Bearing Ratio (CBR) Method but determined at 1/4 in. pennetration and in pounds per square inch.

The characteristics of the soils and mixtures are given in Table 14. The maximum density and related data are given in Tables 15 and 16. The strengths for all the soils and mixtures, after compaction to maximum density by the

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4 Science, Vol. 131, June 24, 1960, pp. 1870-4.
5 Pres., Omaha Testing Labs., Omaha, Neb.
6 Omaha Testing Labs., Omaha, Neb.
7 Omaha Testing Labs., Omaha, Neb.
8 "The Bearing Index as a Criterion for the Maximum Density Requirement," by W. H. Campen, L. G. Erickson, and J. R. Smith, Highway Research Board, 1945.

TABLE 14.—CHARACTERISTICS OF SOIL MIXTURES

Soil		Gradation—Passing Sieve No. in percentage						Liquid Limit	Plasticity Index	Spec. Gr.
No.	Source	270	200	40	10	4	3/4-in.			
1	Omaha, Nebr.	84.0	88.0	100.0	100.0	100.0	100.0	29.5	5.0	2.69
2	Omaha, Nebr.	98.0	100.0	100.0	100.0	100.0	100.0	34.5	10.5	2.7-
6	Kearney, Nebr.	85.0	97.5	100.0	100.0	100.0	100.0	38.0	19.0	2.68
9	Grand Island, Nebr.	48.0	51.4	80.0	100.0	100.0	100.0	24.0	9.0	2.62
10	Waterloo, Ia.		34.6	67.6	94.4	100.0	100.0	24.2	7.7	2.65
13	Scribner, Nebr.	96.8	98.6	99.8	100.0	100.0	100.0	41.7	21.1	2.60
Soil-Aggregate										
No.	Source									
1	Omaha, Nebr.		20.0	30.0	60.0	88.0	100.0	26.5	7.5	2.63
2	Omaha, Nebr.		20.0	30.0	60.0	88.0	100.0	34.0	21.0	2.62
7	Waterloo, Ia.		7.2	24.7	42.5	47.3	100.0	15.2	2.8	2.63
8	Sheridan, Wyo.		13.8	26.2	52.3	61.6	100.0	24.5	13.0	2.60
9	Omaha, Nebr.		11.4	26.3	47.3	71.9	100.0	20.1	4.8	2.61
11	Waterloo, Ia.		11.2	30.1	75.1	91.7	100.0	23.0	5.0	2.64

standard method, are given in Table 17. The strengths of three soils and four soil aggregate mixtures after compaction to maximum density with the three methods are given in Table 18.

The relationship between strength and maximum density for three of the fine grained soils is shown graphically in Fig. 1. The density-energy relationship is shown in Fig. 2, and the strength-energy relationship is shown in Fig. 3.

A study of the data in Tables 17 and 18 warrant the following deductions:

1. The strength of soils and soil-aggregate mixtures compacted to maximum density at optimum moisture by the standard Proctor method varies over a wide range (Table 17) and does not appear to be directly related to gradation, plasticity index, unit weight, or pore space. This substantiates Mr. Mullis' contention that the maximum density is not a good criterion for strength.

TABLE 15.—MAXIMUM DENSITIES AND OPTIMUM MOISTURES WITH DIFFERENT METHODS OF COMPACTION

Soil No.	Densities and Moistures					
	Method No. 1		Method No. 2		Method No. 3	
	Density, in pcf	Moisture, in percentage	Density, in pcf	Moisture, in percentage	Density, in pcf	Moisture, in percentage
1	113.5	15.0				
2	110.0	18.0	112.5	17.0	116.2	15.7
6	107.5	18.1				
9	124.2	10.5				
10	113.7	15.0	117.5	13.2	124.3	10.6
13	101.0	18.5	106.6	17.9	109.2	16.0
Soil Aggregate No.						
1	139.5	6.0				
2	137.5	6.7				
7	140.8	5.8	141.5	5.0	143.5	3.7
8	138.9	5.9	140.4	5.3	142.9	4.3
9	141.9	4.8	142.2	4.8	142.4	4.4
11	130.0	9.4	130.2	8.9	135.2	6.9

The data in Table 18 show that the strength, with any one soil or soil aggregate mixture, increases as the density is increased. However, the increase in strength is not directly proportional to density when all the soils or soil-aggregate mixtures are considered. This is further proof that the strength can not be judged from the density. This point is further emphasized by the curves in Fig. 1, that are constructed from the data in Table 18 for three fine-grained soils.

The curves in Fig. 2 show that the energy required to compact soils is related, in a general way, to the plasticity index, in that the higher the plasticity index the greater the amount of energy required. These data are not directly related to this discussion but are included because it is related to the general subject of soil-densification.

The curves in Fig. 3 show the relationship between energy and strength. They indicate that the amount of energy required to produce a given strength varies over a wide range. For instance, if a strength of 478 psi is required,

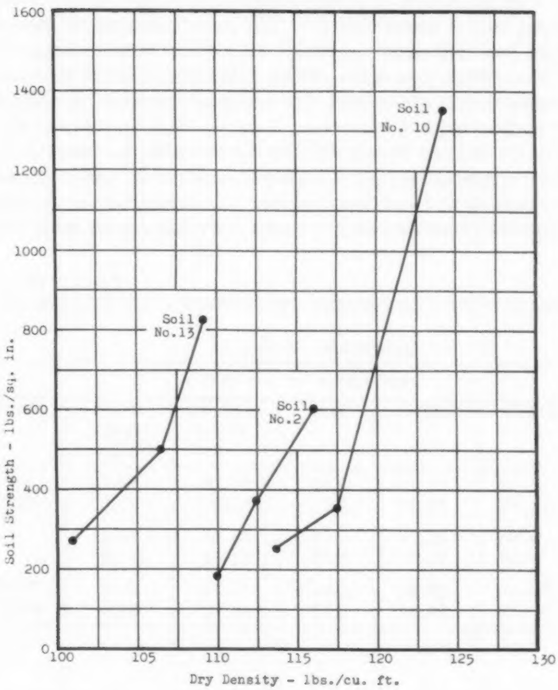

FIG. 1.—DENSITY-STRENGTH RELATIONSHIP

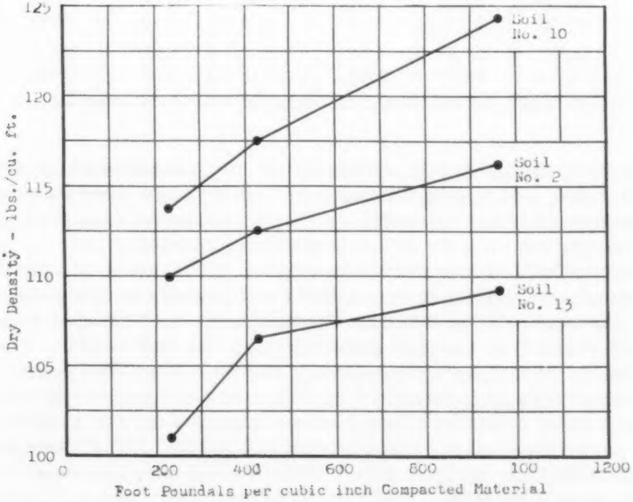

FIG. 2.—DENSITY-ENERGY RELATIONSHIP

the energy per cubic inch of compacted material is 410, 490, and 665 ft-poundals for soils 13, 10, and 2 respectively. The principal reason for including these data is to point out that density, while not a quantitative measure of strength, can be used to control strength. Thus, after the density required to produce desired strength with a given soil has been determined, it can then be used to control field compaction.

2. The data in Tables 15, 16, 17 and 18 can also be used to discuss the effect of bond strength, pore space, and percentage of water-filled pore space. First, it is evident that bond does not control strength. Soil sample 13, Table 17, has the highest plasticity index, that is, a measure of bond strength, but its

TABLE 16.—VOLUMETRIC

Soil No.	Compaction Method #1					Compaction	
	Percentage by Volume					Percentage	
	Solids	Water	Air	Pore Space	Pore Space Filled	Solids	Water
1	67.62	27.28	5.10	32.38	84.2		
2	65.29	31.73	2.98	34.71	92.1	66.77	30.65
6	64.05	31.18	4.77	35.95	86.8		
9	75.97	20.90	3.13	24.03	87.0		
10	68.76	27.33	3.91	31.24	89.0	71.06	24.86
13	62.25	29.94	7.81	37.75	82.9	65.70	30.58
Soil Aggregate No.							
1	85.00	13.41	1.59	15.00	89.4		
2	84.10	14.76	1.14	15.90	92.8		
7	85.80	13.09	1.11	14.20	92.1	86.22	11.34
8	85.61	13.13	1.26	14.39	91.3	86.54	11.92
9	87.13	10.92	1.95	12.87	85.0	87.31	10.94
11	79.52	19.58	0.90	20.48	95.6	79.64	18.57

strength is low. Sample 1, with very low plasticity index, has excellent strength. In the same table, soil-aggregate mixtures 9 and 11 have equal plasticity index, but the former is 8 times as strong as 11. Furthermore, samples 7 and 8 have similar strength but entirely different plasticity indexes.

In connection with this matter of bond strength, it should be pointed out that clean sands and gravels, whether graded or not, possess very high bearing values when thoroughly consolidated. Furthermore, well graded cohesionless mixtures of rounded or angular materials possess high bearing values also.

3. As far as pore space is concerned, all the fine grained soils, Table 16, have pore spaces ranging from 24% to 38%, when compacted by the standard method, and when soils 2, 10, and 13 are compacted by the modified method they have pore space content ranging from 25% to 33%. All of these pore space contents show that normal typical fine grained soils do not have more than 40% pore space after compaction.

Looking now at the soil-aggregate mixtures in Table 16, it will be noted that the pore space ranges from 13% to 16%, with the exception of sample 11, that shows 21%, after compaction with the standard method. Furthermore, when samples 7, 8, 9, and 11 are compacted by the modified method, the pore space ranges from 12.0% to 12.5%. All of these tests, indicate that a maximum pore space of 10%, as required by Mr. Mullis, is not feasible. At this point, attention should be called to the fact that the modified method of compaction involves a compactive effort, that is extremely difficult to duplicate in the field.

Mr. Mullis' requirement for filling 90% of the pore space with water during the densification process is not only too rigid but is not realistic when com-

ANALYSES

Method #2			Compaction Method #3				
by Volume			Percentage by Volume				
Air	Pore Space	Pore Space Filled	Solids	Water	Air	Pore Space	Pore Space Filled
2.58	33.23	92.8	68.97	29.24	1.79	31.03	94.60
4.08	28.94	87.6	75.17	21.12	3.71	24.83	87.00
3.72	34.30	89.8	67.31	28.00	4.69	32.69	87.40
2.44	13.78	82.2	87.44	8.51	4.05	12.56	67.80
1.54	13.46	88.6	87.77	10.04	2.19	12.23	82.00
1.76	12.69	86.2	87.44	10.04	2.52	12.56	80.00
1.79	20.36	91.2	82.70	14.95	2.35	17.30	86.4

pared with actual results. A study of the results obtained with the fine grained soils, Table 16, shows water-filled percentages ranging from 85% to 92% when compaction is done by the standard method. When the modified method is used the percentage ranges from 87% to 95%.

The results shown for the soil-aggregate mixtures, Table 16, support Mr. Mullis' contention in five out of the six samples when compaction is done by the standard method. However, when higher compactive efforts are used, in order to develop less pore space and more strength, the percent of water-filled pore space decreases. This phenomenon is not readily explainable, but the results are not to be questioned. It appears that in order to attain high percentages of water-filled pore space, the densities have to be reduced. This will increase the percentage of water and usually reduce the strength.

Mr. Mullis asserts that probably 90% of the pore space will eventually become filled with water. The writers do not have field data along this line but

do have some laboratory data. In connection with the determination of volume change on drying and freezing, that will be discussed hereinafter, the absorption by capillarity of compacted samples was determined, and in order to show the effect on the saturation, the percent of water-filled pore space was computed. The results obtained with 9 soils and 5 soil aggregate mixtures are shown in Table 19. It will be noted that the percentage of water filled space after absorption ranges from 87 to 95 for the fine grained soils and 90% to 99%

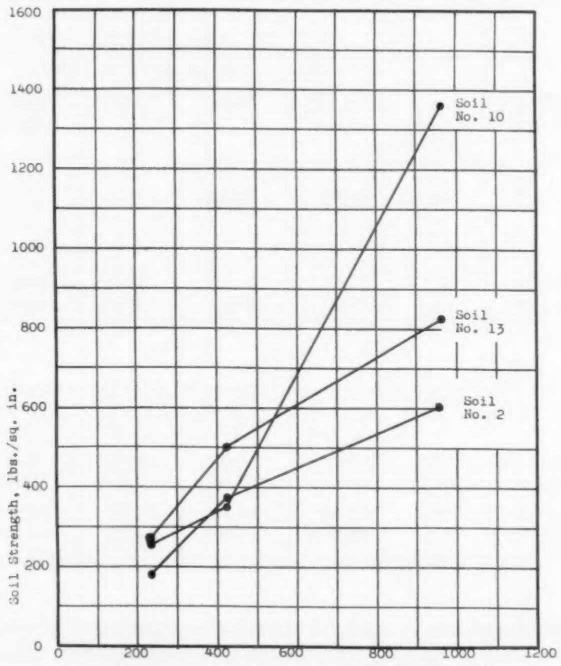

FIG. 3.—COMPACTIVE EFFORT-STRENGTH RELATIONSHIP

for the soil aggregate mixtures. The results indicate that the degree of saturation of the soils and soil-aggregate mixtures varies over a fairly wide range, and of course, corresponding changes will occur in strength. It seems evident, therefore, that the proper procedure for design would be to compact soils and mixtures to as high densities as practicable to reduce pore spaces to practical minimums, and then to submit them to the action of capillary water before determining the strength.

The capillary tests were made by replacing the bottom of the casting mold with a perforated bottom, placing the mold assembly in a pan, adding water until 1/2 in. of the sample was submerged, and storing the pan in a moisture cabinet for 7 days. We believe the conditions of this test are at least as severe as those to be encountered by field installations.

Because Mr. Mullis mentions contraction and expansion in this paper, it is appropriate to include data[9] along those lines in this discussion.

TABLE 17.—STRENGTH OF SCILS AND SOIL-AGGREGATE MIXTURE AT STANDARD MAXIMUM DENSITY

Soil No.	Strength psi	Soil Aggregate No.	Strength psi
1	710	1	334
2	185	2	643
6	407	7	137
9	707	8	111
10	258	9	1,273
13	278	11	159

TABLE 18.—STRENGTH AT VARIOUS MAXIMUM DENSITIES

Soil No.	Max. Density, in pcf	Strength, in psi	Soil Aggregate No.	Max. Density, in pcf	Strength, in psi
	110.0	185		140.8	137
2	112.5	376	7	141.5	834
	116.2	605		143.5	2,419
	113.7	258		138.9	111
10	117.5	353	8	140.4	390
	124.3	1,362		142.9	573
	101.0	278		141.9	1,273
13	106.6	501	9	142.2	1,416
	109.2	828		142.4	1,974
				130.0	159
			11	130.2	185
				135.2	1,054

Ten soils and six soil aggregate mixtures were studied by the writers. The plasticity index of the soils ranged from 5 to 52 and that of the soil aggregate mixture from 6 to 21.

The soils were compacted to near maximum density and optimum moisture by the standard Proctor method and were then expelled from the mold. Shrinkage on drying was determined by measuring the maximum change in diameter at room temperature. The volume change on freezing was determined by coating the plugs with paraffin wax and then submitting them to a temperature of 10° F below zero for 6 hr.

9 "Some Physical Properties of Densified Soils," by W. H. Campen, L. G. Erickson, and J. R. Smith, Highway Research Board, 1942.

TABLE 19.—ABSORPTION BY CAPILLARITY AND RELATED DATA

Soil No.	Compaction Data		Volumetric Analyses—Percentage by Volume						
			Before Absorption					After Absorption	
	Dry Density, in pcf	Optimum Moisture, in percent	Solids	Water	Air	Pore Space	Pore Space Filled	Air	Pore Space Filled
1	113.3	15.5	67.50	28.14	4.36	32.50	86.6	2.05	93.7
2	125.0	10.2	76.46	20.43	3.11	23.54	86.8	2.67	88.6
3	111.5	16.5	66.18	29.49	4.33	33.82	87.2	3.79	88.8
4	115.3	13.9	70.80	25.69	3.59	29.20	87.9	1.98	93.2
5	110.0	18.1	66.02	31.91	2.07	33.98	93.9	1.54	95.5
6	105.8	17.5	64.96	29.67	5.37	35.04	84.6	2.83	92.0
7	115.9	14.7	70.09	27.30	2.61	29.91	91.2	1.96	93.5
8	107.5	18.1	64.28	31.18	4.54	35.72	85.0	3.25	90.8
9	105.5	20.0	63.32	33.81	2.87	36.68	92.2	2.19	94.0
Soil Aggregate No.									
1	139.5	6.0	85.00	13.41	1.59	15.0	89.4	0.94	93.8
2	136.0	6.5	83.19	14.17	2.64	16.81	84.4	1.33	89.5
3	140.3	5.8	86.15	13.04	0.81	13.85	94.2	0.16	98.8
4	137.0	6.9	83.80	15.15	1.05	16.20	93.5	0.52	96.8
5	138.6	5.9	85.43	13.10	1.47	14.57	89.9	0.60	96.0

The results may be summarized as follows:

1. All mixtures shrink on drying and the shrinkage generally increases as the optimum moisture increases. The lineal shrinkage of the fine grained soils ranges from 0.5% to 4.3%, while that of the soil aggregate mixtures from 0.1% to 0.5%.

2. The fine-grained soils shrink on freezing, and the shrinkage increases as the optimum water content increases, whereas the soil-aggregate mixtures expand and the expansion increases as the ratio of water used at optimum to the computed optimum water content increases. The fine grained soils shrink from 0.1% to 1.0% and the soil-aggregate mixtures expand from 0.2 to 0.5%.

As has been noted, the strength of the soils in our research has been determined by a method similar to the CBR Method. In the writers' opinion this type of method is the proper one to use because it is suitable for all types of soil and soil-aggregate mixtures, from very plastic ones to noncohessive ones. The test measures resistance to consolidation and displacement, both of which determine the ability to sustain vertical loads. The Hubbard Field Test on the other hand, measures resistance to shear that is dependent on cohesion to a large extent. The test was not intended for cohesionless mixtures.

It is important to note that the ductility required is proportional to the maximum elastic strains in the material. For materials other than mild steel, the required ductility may be higher. As an example, for the strong 7075-T6 aluminum alloy, the same conservative estimate yields about 6%, and a similar amount is needed for the steels of very high strength now being used in aircraft. It will be seen that the larger the elastic range in terms of strain, the more ductility is required.

Most acceptable structural materials in present use will then have an ample reserve of ductility so as to be unimpaired in strength by stress concentrations at the connections. Only in materials of low ductility will such stress concentrations affect the strength. Such occasions can arise with large forgings where the ductility in some directions may be low. Plates loaded in tension through the thickness have sometimes given rise to unanticipated failures of this kind.

We have spoken earlier of a trend in structural practice to design in terms of the load at failure of the structure. Design on this basis must take account of plastic behavior. On this basis redundant structures are found to possess reserves of strength not shown by statically determinate structures. This point of view dates back to the beginning of the 20th century, but has received explicit formulation by J. A. Van den Broek[2] and more recently by other engineers, in particular J. F. Baker.[3] To a large extent this attitude represents the philosophy of the aircraft designer who deliberately seeks to predict the actual behavior of this structure at failure. Because of the economics of aircraft construction, the aeronautical engineer has the advantage of frequent tests to destruction of his assemblies, so that he can be fully aware of the modes of failure of his designs. The civil engineer on the other hand is justifiably reluctant to depart from well-established practice until a sufficient volume of test results has been accumulated to give him confidence. Many such tests, however, are under way and enough data have been obtained to show that the point of view of limit design will have important effects on the design philosophy of civil engineers.

Limit design often leads to different factors of safety than would be predicted from conventional elastic design. A rolled shape tested as a simply-supported beam will collapse when the flanges have become fully plastic at the point of maximum bending moment. This condition is reached for normal shapes at about 115% of the bending moment which produces first yielding. It is assumed, of course, that the beam is adequately supported against premature failure by lateral instability. As the moment distribution is unaffected by yielding in a simple beam, elastic design and limit design in this case lead to the same proportions and the same factor of safety. The situation is quite different, however, in a fixed-end beam under uniform load. Here the greatest bending moments under elastic conditions are at the built-in ends and yielding begins first at these points. When the ends have reached their maximum plastic moment, the rest of the beam is still elastic and more load can be applied without large increase in deflection, the end moments remaining constant and the center moment increasing. Collapse finally occurs when the bending moment at midspan reaches its full plastic value. Elastic design ignores the action beyond the point at which the end moments develop plastic behavior. Limit design, on the other hand, considers the redistribution of bending moment that

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[2] "Theory of Limit Design," by J. A. Van den Broek, Wiley, New York, 1948.

[3] "The Steel Skeleton," Vol. II, by J. F. Baker, M. R. Horne, and J. Heyman, Cambridge Univ. Press, New York, 1956.

DEVELOPING ALTERNATIVE URBAN TRANSPORTATION SYSTEMS[a]

Discussion by Keneth M. Hoover, S. D. Forsythe, and Nathan Cherniack

KENETH M. HOOVER.[2]—Mr. McConochie's conclusions seem to be slanted toward the acquisition of rights of way for future highways alone. The writer thinks that acquisition of rights of way for transit were just as important and that this was the intent of the majority who worked on the program.

More important to the writer is the fact that here, for the first time, an objective study was made to ascertain the total transportation requirements of a metropolitan community and that the study resulted in a plan that brought about action to organize the community to cope with the problem.

Because Mr. McConochie wrote this paper, Congress has taken action, and the President of the United States has signed a bill providing for the creation of a Federal agency to modify and carry out the basic findings of the study. The writer has no doubt that this agency will enlarge and refine the findings of the planners, economists, engineers, and others who conducted the present study.

S. D. FORSYTHE.[3]—The author very ably condenses the great mass of data gathered in connection with the Transportation Plan for the National Capitol Region, so that the important considerations offered in support of each of the four plans are brought into reasonably sharp focus. The conclusions reached by the planning groups represent a middle ground between the two extreme plans, the Auto-Dominant Plan and the All Rail Rapid Transit Plan with Recommended Systems of Highways, and as such (that is, a middle ground), it must be conceded that their position is sound. However, the thoughts expressed that "Express buses would be used in the early years on the routes ultimately to be served by rail rapid transit," and that "Rail Rapid Transit to certain sections of the area was shown to be justified as the ultimate plan by estimated patronage" is disturbing and ambiguous. It suggests that either the authors of the plan look upon rail rapid transit (and by this term it is assumed they include all types of multiple car operation) as a last resort that may never have to be installed at all on these routes, or else, if they do expect rail rapid transit to be installed by 1980, on these routes, they have not considered the capital investment in the so called second and third stages (as illustrated in Fig. 3) that must be written a short span of years. Neither have they considered the staging problems attendant upon maintaining an express bus line operating at or near capacity while converting the median strip or private right-of-way from bus roadways and stations to rail rapid transit facilities.

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[a] March, 1960, by William R. McConochie.
[2] Chf. Engr., San Francisco Bay Area Rapid Transit District, San Francisco, Calif.
[3] Genl. Supt. of Engrg., Chicago Transit Authority, Chicago, Ill.

If the planners are satisfied that some form of rail rapid transit will be justified on some routes by 1980, then it seems wiser to plan immediately to install those facilities at the earliest possible time than to plan three different periods of intense and expensive construction before the ultimate plan is realized.

The ability of modern rapid transit to attract riders is well illustrated by the Congress Street (expressway) route in Chicago, Ill. Today, this route, serving the same territory the old Garfield Park elevated line served with competing rapid transit lines on either side of it, now as then, is carrying 50% to 65% more passengers in the rush hour periods than the old route.

Furthermore, this line operating between 30% and 40% of capacity is carrying 150 people in the peak periods for every 100 people the adjoining eight lane expressway, operating at between 80% and 90% capacity, is carrying in the same periods. On the basis of investment per passenger capacity, rail rapid transit facilities should prove much less costly than the three stage plan proposed.

One last point seems worthy of further thought. Various types of multi-car rapid transit systems are being given more intense study in the United States and in Europe at the present time than at any previous time in this century. Washington D. C., is in an excellent position to profit by, and indeed to contribute greatly, to these studies. Therefore, it seems quite premature to assume that articulated rail cars will prove to be the most satisfactory rapid transit vehicle to use in Washington, where there is no compulsion to conform to any existing types. As the nation's capitol, it should be equipped with the finest type of system the nation can produce, and it is to be earnestly hoped that the same combined effort on the part of so many able people will characterize the actual building and operating of the adopted plan.

NATHAN CHERNIACK,[4] F. ASCE.—Mr. McConochie presents a case study on urban transportation for a given metropolitan area, Washington, D. C. and its environs. Its value is enhanced considerably by the fact that it consists of several distinct studies of alternate methods that might solve the transportation problems of metropolitan Washington. Each separate solution involves its own economic cost with correlative physical advantages and disadvantages.

There have been similar urban transportation studies for specific cities to meet their respective needs. Few, however, have been reported in technical journals with the view of distilling out general urban transportation principles. Out of the urban transportation study for metropolitan Washington, McConochie has made two contributions. He has sharpened up the technical language, something sorely needed before the controversial issues involved in the general topic of urban transportation can be resolved. He has also indicated some general principles that the engineering profession would do well to adopt in appraising urban transportation needs in other metropolitan areas.

For example, the author states that "Throughout this paper, the terms 'express' and 'rapid transit' are used interchangeably to indicate services by buses on freeways and parkways, as well as rail vehicles on private rights of way." From current literature on urban transportation, one gets the impression that there are only two choices possible for meeting urban transportation needs. One is to adopt transportation on the highways in autos, with very limited ca-

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[4] Economist, Port of N. Y. Authority, New York, N. Y.

pacities to handle people in rush hours; the other is to make use of rapid transit on rails on private rights of way, with tremendously high capacities to handle people in rush hours, even if population densities in tributary areas along the rail alignments are such as to preclude the possibility of ever utilizing a fraction of the designed capacities. The possibility that there is a third choice, that much more widespread use could be made of express buses on freeways and parkways having sufficiently high passenger carrying capacity to rapidly transport masses of people in rush hours, appears to be completely absent from current literature on urban transportation.[5] McConochie's treatment of the terms "express" and "rapid transit" as interchangeable and as describing passenger transport services either by rail vehicles, or private rights of way, or by buses on freeways and parkways is indeed refreshing and reflective of advanced thinking on modern urban transportation.

After analyzing the Auto-Dominant plan for which autos would meet "the major portion of the travel needs of the metropolitan area," and "public transportation in the form of local buses operating on the surfaces streets giving service of the type and relative extent provided in 1955," as "transit playing a minor role," McConochie comes to these conclusions with respect to the Washington metropolitan area. "There are serious engineering questions as to the practicability of the autodominant plan." "A transportation policy requiring the National Capital Region to be served almost exclusively by automobiles would be incompatible with the distribution of living, shopping, and working places of the area's inhabitants as foreseen by the staff of the Commission and Council."

In effect, the author is saying that to set up an auto-dominant plan for any urban area is to set up a "straw man." Few large urban areas could get along without a substantial proportion of their rush hour passenger movements being handled by some type of mass transit.

On the other hand, the author points out "that a good rapid transit system (as he defines it) can complement a network of urban freeways, but it cannot provide an adequate substitute for modern highway facilities." "Even an over-designed system of rapid transit facilities would not drastically reduce the need for freeways." "This highway system (an enlarged highway network) would be necessary in connection with either the all-bus or the all-rail transit plan." "The rapid transit system involving the lowest investment for special transit facilities would be comprised of express bus routes on freeways built for general traffic, with separate lanes for buses where needed."

The recommended Plan IV recognizes the need for mass transit but does not recommend all-rail transit exclusively. It recommends 66 route miles of express buses on freeways and parkways and 34 miles of rail rapid transit to meet metropolitan Washington's transit needs of the future. Because an enlarged freeway and parkway system is essential even with a transit plan, why not utilize the freeways and parkways exclusively, as a modern transit system with express buses? Why recommend the sinking of additional new capital in complementary over-designed rail transit system when surely, for most of the proposed rail transit mileage, the anticipated rush hour passenger density in the next two decades would not demand a fraction of the available rail transit capacity that would be provided? At the same time, express buses on freeways would meet all the transit needs for the next two decades.

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[5] "Passenger Data for Urban Transportation Planning," by Nathan Cheriak, Proceedings, ASCE, Vol. 85, No. HW 4, December, 1959.

The author, in fact, states that "Consideration was given to bus operation on private roadways in a wide central mall of the north central freeway as a long term or even permanent solution to the rapid transit requirements of the area." He goes on to say that "There would be capacity for buses to operate in this manner (having only one-half or one-third of the buses stop at any one station; the balance pass that station as non-stop or skip-stop buses) to at least 1980, on the freeways proposed for rapid transit bus service in the recommended scheme - Plan IV." But he says further, "it must be constantly kept in mind that even the target date of 1980 is a relatively short time objective in a region that may continue to grow in population for decades or even centuries." True!

Instead of recommending the sinking of new capital in rail transit now, as incorporated in Plan IV, why was the sounder recommendation not incorporated in Plan IV? There, McConochie states, that "it is recommended that in planning future radial freeways, a cross section similar to that shown in Fig. 3 be provided to afford maximum flexibility and to provide reserve capacity for automobile and truck traffic, as well as for the mass movement of people." "In the fourth stage of development, the bus roadways could be displaced by a double track rail rapid transit facility which would provide capacity for any volume of transit patronage likely to develop within the next century."

Hence, the final query: Why recommend the building of a rail transit system now, in the first stage, when it is possible to so plan freeways and parkways for stage construction that rail transit could be provided when a real need has been conclusively demonstrated through usage of the freeways and parkways in the several stages, by express buses, preceding the final conversion to rail transit?

ANALYZING AND PROJECTING TRAVEL DATA[a]

Discussion by Robert L. Morris and James S. Burch

ROBERT L. MORRIS,[6] M. ASCE.—The writer finds Mr. Wilbur Smith's article enlightening as "one of the most exhaustive analyses of urban travel ever undertaken." No doubt it is, and those who use the results of this study should be grateful for the care and detail that were given to it.

The only serious criticism lies with the reference to the Potomac River as a "psychological barrier." It is difficult to see what subconscious blocks this particular river inflicts that others (such as the Anacostia, also within the District of Columbia) do not. One is led to suspect that such terms are but an attempt to camouflage the inadequacies of the "interactance formula" to handle such situations. For, essentially, Mr. Smith's technique involves the same flaw that is inherent in all methods of projecting existing travel patterns to determine future highway demands. That is, they do not take into consideration the latent demand for cacilities that have no comparable or satisfactory existing route. Few interviewees, in response to an Origin-Destination questionnaire, will indicate a desire to go to a site that is presently difficult to reach for any of a number of reasons. Yet by now, one should be well aware of the lesson taught by the avalanche of vehicles that descends on so many new facilities as soon as they are opened. With more bridges due to be constructed over the Potomac, who doubts that they will quickly be utilized to capacity?

Patrick Henry's lament that he had "but one lamp by which my feet are guided and that is the lamp of experience," applies equally to the traffic engineer, for with all the available theories, there exists no means of determining future travel patterns except by basing them on past trends and habits, knowing full well such projections are frequently grossly inadequate.

The solution to this dilemma would seem to lie within Mr. Smith's grasp. The gravity model, on which the interactance formula is based, offers the key that is unavailable to other existing systems of projection. Mr. Smith recognizes that either travel time or mileage can be used as an expression of trip length and even considers "airline" miles. In any such study area, particularly one containing extensive physical obstacles, it would seem that a first step would be to make a purely theoretical determination of interzonal traffic demand, using the gravity model approach. Trip attraction and production would be functions of residence, employment, auto ownership, and so forth. The fundamental difference would be in the measure of trip length, in which airline distances would be used, ignoring all existing highway facilities. The result would indicate the latent demand for vehicular routes. Over this pattern would

[a] June, 1960, by Wilbur S. Smith.

[6] Senior Planner, Transportation, National Capital Downtown Committee, Inc., Washington, D. C.

be superimposed the existing transportation structure, instantly revealing the shortcomings and their relative significance. The procedure from this point could follow Mr. Smith's technique, with all "psychological barriers" removed.

JAMES S. BURCH.[7]—It is quite evident that Mr. Wilbur Smith has made a most exhaustive and valuable study and projection of the traffic demands of the future in the Washington, D. C., area. One of the primary aspects of this, and other modern studies is the coordinate work of specialists in other fields such as sociology, city planning, and economics.

Modern highway and street facilities are needed to replace and augment obsolete facilities. They are very expensive and must be located and designed adequately for long traffic service life. Largely because of the fact that space is not available and is almost prohibitive in cost, such facilities involve very high capital investment. The cost is extreme where traffic interchanges or river bridges are involved.

To plan for long traffic service life requires an estimation of the expected traffic nature and demand some twenty to thirty years in advance. Such estimation is a very complex problem, since traffic demand involves an infinite variety of purposes in a growing and changing population and economy, changing urban boundaries and traffic direction, and with no price tags on the use of the facilities. Simple projections by extrapolation are inadequate. Acceptable methods of forecasting traffic use of a given facility twenty years hence is a complex and continuing effort of those technicians who conceive and plan these facilities. Basic data are required, but are seldom immediately available. There are no "ticket" records on even the present use of the passenger car.

Within the last two decades, traffic has not only shown phenomenal growth, but has changed greatly in characteristics and pattern, especially with the changing nature of the urban area.

These urban changes include the outward "explosion" of the city; the shorter workday and workweek; the two-car family; dispersion of traffic generators and trip termini clusters, (such as work sites, schools, churches, and supermarkets); decreasing dependance on mass transit; shopping in suburban satellite store groups and store branches; shopping in the evening; the lowering of car occupancy; and the modern "pressure" to save travel time and find parking space. In Washington, especially, there has been an abnormal turnover in residents since 1950. Even the permanent residents have moved to the suburbs and nearby small towns by the scores of thousands. With all these chaotic changes occurring simultaneously in all cities within a decade or two, traffic evaluation and projection has been a very precarious problem.

However, certain basic travel characteristics appear to have become stabilized, and to assume an increasingly more important position as a basis for projections. These include the vast preponderance of "home" based trips, the increasing importance of "work" as a motor vehicle trip purpose, the fact that the majority of urban trips are quite short--in the order of 1 to 5 miles; lowered car occupancy, and the coincidental occurrence of the home-work and work-home movements with the daily peak traffic hours.

With these elements now appearing to have become somewhat stabilized, there are evidences that there may be other aspects in which more orderly growth rates may be anticipated. It now (1900) appears, after fifteen post-war years during which the motorist seemed neither to know nor care about the

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[7] Planning Engineer, North Carolina State Highway Commission.

actual cost of use of his car, that he is now becoming "dollar conscious," with the emphasis of his interest now appearing to center on vehicle cost, depreciation rate, and gasoline mileage. Witness the great upsurge in purchases of the small and compact or "economy type" passenger cars.

The traditional 400-mile weekend pleasure trip habit seems to be abating in its rate of growth. With modern home appliances, air-conditioned homes, television and back yard cooking, the motorist and his wife are discovering the relaxing pleasures of staying at home. The great popularity of boating on nearby rivers and lakes has vastly stimulated family boat ownership, and this is having a dampening effect on weekend highway travel. Scheduled airlines are absorbing much of the weekday travel involving trips of over 150 miles. For the first time, the annual travel per car is moving downward from the traditional 10,000 miles, with the peak value of this ratio appearing to have already occurred.

There is much evidence that within a very few years, we will be approaching the point of saturation in the cars-per-family and the cars-per-employed-person ratios. This does not mean that car registration will not grow, but merely that the growth may more nearly parallel, rather than greatly exceed, the rate of population and employment growth. In the State of North Carolina, curves showing these ratios are definitely tending to diminish in growth rate. The same is true for total gasoline consumption, now (1960) at little over 2%. In one recent year, there was actually a reduction in the state-wide motor vehicle registration (due to a combination of circumstances--but it did happen).

The urbanization trend in America is marked, not only by migration to the great cities, but also by the enlargement and spread of the small cities and larger towns. As these latter enlarge, they grow towards each other, tend to join and to form multi-centered urban complexes. The economic and sociological affiliation between such towns is already strong. It is often easier to live in one town and commute to another, than to live on one side and work on the other side of the same town. Also, it is often easier to shop in the nearby town than to drive into one's own downtown business district. This trend toward the multi-centered community may have a profound effect on the problem of traffic volume projections and traffic facility planning in such areas.

For this reason, the traffic phase of a special study has been recently completed in North Carolina involving the evaluation of traffic "interactance" between five nearby but independent cities and towns which have strong mutual attractions for each other. This study involved a special origin-destination traffic survey of 118,000 roadside interviews at 35 stations.

One of the purposes of the study was to refine the basic gravity model formula for traffic production, involving only the variables of population and distance, as it might best be applied to such a situation. With this data, by analytical statistics, we were able to develop a quadratic formula relationship involving the population of each pair of places in the numerator, and the distances between them in the denominator, resulting in a high level of confidence as checked against several studies in other areas. This formula could then be applied to any normal situation to develop the increment of traffic demand between any two communities. The summation of these overlapping increments would be indicative of the total traffic demand at any point.

To adapt this formula for use in traffic demand forecasting procedures, population specialists made long-range projections of 1980 community populations all over the state. With distances between community centers being fixed, it was possible, with a digital computer program, to evaluate these increments

of future traffic demand, and to summarize the totals of increments for each highway segment in the state for the year 1980. These traffic demand projections became basic to decisions on classification of statewide systems, design standards, long-range improvement plans and future needs. The selection, design, planning, priority and financing of highway construction projects are now following these schedules in the entire state.

Summary.—Long-range traffic demand projections are most complex, in that they are the resultant of a vast array of economic, sociological and geographical forces, being ultimately determined by the decisions of drivers who have absolute freedom of choice as to time, direction, frequency, speed, route, and occupancy. In this sense, such studies are similar to market analyses and their projections.

However, most products and services involve flexibility as to quantity, style, time, price and inventory. Highway and street facilities are absolutely fixed and permanent, with almost no possibility of adjustment to demand, At the same time, they represent perhaps the most expensive investment in public works. The demand simply must be projected and evaluated as a precedent to construction decisions, no matter how complex.

HIGH TEMPERATURE EFFECTS ON BITUMINOUS MIXES[a]

Discussion by Bob M. Gallaway

BOB M. GALLAWAY.[14]—Mr. Gotolski is to be commended for the very fine manner in which he has presented the results of his study, however, it is at the same time regretable that the research was not more extensive and more general in its application. In his conclusions (item 5) one may believe that an asphalt (any asphalt) may be safely and properly heated to 375°F during the mixing process and held without damage at or near this temperature through the field placing operation. There is a tremendous wealth of data on this very subject and a review of these data should leave no question in one's mind of the grave danger of overheating any grade of paving asphalt when it is exposed in the thin films present in most hot-mix asphaltic concrete designs.

Recent research by the United States Bureau of Public Roads (BPR) revealed the wide variation in viscosities of 85-100 penetration asphalts heated to 275°F. Well over 100 different asphalts were included in the study. The Asphalt Institute is fully aware of the differences in paving asphalts that meet the same specification requirement as to penetration, softening point and ductility. The viscosity susceptibility differences are taken into account by their recommendation that proper mixing temperature be taken from a plot of data relating viscosity and temperature.

Neither of these organizations based their conclusions on one or even a dozen asphalts. The proper handling of an asphalt, temperature-wise, is surely not an arbitrary affair. In the first place penetration, at best, should be used only as a manufacturing control and never for a study of fundamental properties. For example, three 85-100 penetration asphalts of different source were heated to 325°F in gallon lots for 30 days and one of these asphalts did not change penetration at all, whereas the other two retained less than half of the original penetration. A refiner may hold tankage of asphalt at 300°F to 350°F for weeks and little change occurs. On the other hand, the same asphalt exposed in thin films to free circulation of air may be rendered useless as a cementing agent for paving mixtures in a matter of minutes.

Test specimens for stability control measurements may be formed successfully by use of the Marshall hammer but to depend on this method of sample formation and stability data therefrom for a basis of conclusions concerning fundamental changes in asphalt cement is quite another thing.

Were extractions made on the mixtures studied, or were the rate of aging conclusions based on the characteristics of the asphalt heated in 5-gallon lots? Were estimates made on the effective film thicknesses of the two different mixes? Would this not have a decided effect on the viscoelastic properties of

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[a] September, 1960, by William H. Gotolski.

[14] Professor, Civ. Engrg., Texas A. & M., College Station, Tex.

the test specimens? Grading as well as asphalt content materially affects workability.

Granted that the conclusion on mixing temperature was qualified, it is misleading, especially in the face of all the data that show the hardening effects of heat and air on thin (7 to 15 microns) films of asphalt on aggregates.

Modern instruments and techniques are available to study changes in the fundamental properties of bituminous materials and even with the best of these the job to be done is tremendous. One with the author's energy and drive in the research field would do well to use the most modern methods and tools in asphalt studies. And, too, there is still a great demand for newer and better tools and forward looking ideas.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411 (SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478 (HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487 (CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580 (EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

SEPTEMBER: 2588(IR3), 2589(IR3), 2590(WW3), 2591(IR3), 2592(HW3), 2593(IR3), 2594(IR3), 2595(IR3), 2596 (HW3), 2597(WW3), 2598(IR3), 2599(WW3), 2600(WW3), 2601(WW3), 2602(WW3), 2603(WW3), 2604(HW3), 2605(SA5), 2606(WW3), 2607(SA5), 2608(ST9), 2609(SA5)[c], 2610(IR3), 2611(WW3)[c], 2612(ST9)[c], 2613(IR3)[c], 2614(HW3)[c].

OCTOBER: 2615(EM5), 2616(EM5), 2617(ST10), 2618(SM5), 2619(EM5), 2620(EM5), 2621(ST10), 2622(EM5), 2623(SM5), 2624(EM5), 2625(SM5), 2626(SM5), 2627(EM5), 2628(EM5), 2629(ST10), 2630(ST10), 2631(PO5)[c], 2632(EM5)[c], 2633(ST10), 2634(ST10), 2635(ST10)[c], 2636(SM5)[c].

NOVEMBER: 2637(ST11), 2638(ST11), 2639(CO3), 2640(ST11), 2641(SA6), 2642(WW4), 2643(ST11), 2644(HY9), 2645(ST11), 2646(HY9), 2647(WW4), 2648(WW4), 2649(WW4), 2650(ST11), 2651(CO3), 2652(HY9), 2653(HY9), 2654(ST11), 2655(HY9), 2656(HY9), 2657(SA6), 2658(WW4), 2659(WW4)[c], 2660(SA6), 2661(CO3), 2662(CO3), 2663(SA6), 2664(CO3)[c], 2665(HY9)[c], 2666(SA6)[c], 2667(ST11)[c].

DECEMBER: 2668(ST12), 2669(IR4), 2670(SM6), 2671(IR4), 2672(IR4), 2673(IR4), 2674(ST12), 2675 (EM6), 2676(IR4), 2677(HW4), 2678(ST12), 2679(EM6), 2680(ST12), 2681(SM6), 2682(IR4), 2683(SM6), 2684(SM6), 2685(IR4), 2686(EM6), 2687(EM6), 2688(EM6), 2689(EM6), 2690(EM6), 2691(EM6)[c], 2692 (ST12), 2693(ST12), 2694(HW4)[c], 2695(IR4)[c], 2696(SM6)[c], 2697(ST12)[c].

c. Discussion of several papers, grouped by divisions.

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PART 2

DECEMBER 1960 — 47 NO. HW4
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE HIGHWAY DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE HIGHWAY DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
HIGHWAY DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

December, 1960

THE FEDERAL HIGHWAY PROGRAM

'Bungle' --- or Blessing?

In the July, 1960, issue of The Reader's Digest an article entitled "Our Great Highway Bungle" makes the claim that "haste, waste, mismanagement and outright graft are making a multibillion-dollar rathole out of the Federal Highway Program." We are told that whereas the Act of 1956 was a "beautiful . . .dream," it has "become a nightmare: of recklessness, extravagance, special privilege, bureaucratic stupidity and sometimes outright thievery." These charges were backed by descriptions of three interchanges in "a sparsely settled area of Nevada;" a "superstreet. . .being driven through the heart of the city" of Omaha; a six-lane highway "driving a 300-foot swath right through town" in Wilmington, Del.; the "projected four-and-a-half-mile road through the center of town." Macon, Ga., "which will bisect an attractive residential district."

In an attempt to answer these charges in some small measure, Commissioner John O. Morton of the New Hampshire Department of Public Works and Highways, Chairman of the ASCE Highway Division, has written the following letter to the author of the article, and to the publisher of the magazine. We are proud to reproduce it here:

"Mr. Karl Detzer
c/o Reader's Digest
Pleasantville, N. Y.

Dear Mr. Detzer:

If I had not spent over thirty years of may life in highway construction work, had not experienced close contact with all grades of employees in New Hampshire's Public Works and Highways Department, had not known intimately and served on committees with many of our country's leading State Highway officials, and had not learned the risk of believing every word and phrase that found its way onto the printed page, I might, as a taxpayer, have been sorely distressed and alarmed by the "Highway Bungle" article in the July issue of Reader's Digest.

Note.—No. 1960-47 is Part 2 of the copyrighted Journal of the Highway Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HW 4, December, 1960.

Because of the unbelievable size of the nation's highway program ever since the 1956 Federal-aid Highway Act was passed by Congress, because of its high estimated cost ($45 billion), there certainly is little reason to feel that the majority of our citizens can be expected to conceive what this amount of money will buy in benefits to the nation, states, communities or the individual. Also, for these very same reasons, it is not only understandable but healthy and justifiable that the American taxpayer should continue to be alert and carefully examine this expensive program with most critical eyes.

A comprehensive, fair, and honest treatment of any program demands a clear, unbiased view of the picture from all sides. When isolated evidences of mismanagement are pointed up, when the misleading title of an article highlights a narrow, myopic focus on a great, nationally beneficial project, when sole emphasis is placed on wholesale "recklessness, extravagance and bureaucratic stupidity," when insinuations are made of grand scale corruption, irreparable damage can result.

At present, the entire cost of this world's greatest public works project is paid for by the highway-user, and until every citizen in America comes to realize that a first-rate national transportation network will benefit him personally, the full impact of the need and importance of this long-needed highway system will not be felt.

Instances of error in judgment, corruption, and misdoings, are bound to appear in such a program where so great a number of people are involved. This has always been so, as you must well know, from the beginning of history where man is concerned, and has not been limited to public works programs but has delved into municipal and governmnet administration, small and big business, even into ecclesiastical affairs.

The remarkable fact is that few instances have occurred in this country's road building program. It is easy to magnify and easy, by innuendo, to multiply those relatively few occasions instead of showing the whole picture where the high majority of great accomplishments would be obvious.

A House Committee report on the Federal-aid Highway Act of 1956 stated two facts that were basic, recognized and agreed upon by all concerned:

1. "The whole economy of the United States is directly dependent upon motor vehicle transportation."
2. "We are failing to keep our highway systems adequate to meet our needs and the backlog of deficiencies required to be overcome has and is constantly piling up at an alarming degree. Unless drastic steps are taken immediately we will fall further and further behind and traffic jams will soon stagnate our growing economy."

New Hampshire's three turnpikes, the 40 mile F. E. Everett (1957) and the 40 mile New Hampshire Coastal (1950) with its Spaulding extension (1957), are fairly accurate barometers of this state's heavy traffic volumes. In one year, 1959, the 15 mile coastal turnpike bore 5.2 million vehicles, 200,000 more than in its first three years and two times the amount forecast for 1959.

Across the nation from 1956 to 1971 it is expected that highway travel will increase nearly 70 per cent, from 623 billion vehicle miles to more than a trillion.

In the 41,000 mile National Interstate and Defense System of Highways is a plan that will greatly increase the traffic-bearing capacity of our entire highway system. It is on this system that the future economy of the nation will be largely dependent. Its manifold benefits you could have explained to the taxpayer and performed a great service.

A first-rate national transportation network will benefit the individual as much as the state or the country. You certainly are aware of how express routes around cities (Route 128 around Boston) have quickly stimulated the growth of industry, shopping, and residential areas, giving greater mobility and job opportunities.

The New York Thruway, adjacent to your magazine's office building, has attracted 650 million dollars worth of industrial, commercial, and residential development along its whole length.

You could have turned the picture again and written about increased traffic safety on these new Interstate highways, how modern design standards, including controlled access, can sharply reduce highway accidents, how the death rate on most of the country's toll expressways last year per one hundred million vehicle miles of travel was about one-third the national rate on rural roads. Safety, the human life factor, is perhaps the greatest of all benefits of these new highways.

If you had written a fair treatment of the Federal-aid Highway Program, all sides of the road-building picture would have been shown. Criticism is justified when wrong doing is evident and every means possible should be taken to correct it and prevent its recurrence.

The Federal-State relationship in the highway program, in addition to assuring high construction standards has always contained checks and counter-checks built in for protection of public money. If it were not for the Bureau of Public Roads' engineering inspections and final auditing of each Federal project, any wrongdoing probably might not have been uncovered. Since 1957 the Bureau has referred to the Department of Justice more than a dozen cases of irregularity.

It continues to seem to me that it is a credit to the Bureau and the State Highway Departments that this program has had so little "bungle." It could be disastrous if this joint undertaking involving thousands and thousands of conscientious, honest, dedicated engineers, administrators and contractors, as well as the economy of our country and safety of our traveling public were to suffer irreparably, merely because the misleading, specious, gossipy, selfish-interest sides were shown instead of the entire, full dimension, truthful picture.

Sincerely,

/s/ John O. Morton
Commissioner.

JOM/lds
Copy to DeWitt Wallace, Publisher
Reader's Digest
Pleasantville, N. Y."

COMMITTEE NEWS

Executive Committee

The present make-up of the committee is as follows: W. A. Buggee, Chairman; J. P. Buckley, Vice Chairman; J. O. Morton, Walter Tacke, T. J. Frater, Members; and C. F. McCormack, Secretary.

T. J. Fratar, Ex Officio; and C. F. McCormack, Secretary.

Mr. Arch N. Carter completed his term of office with the committee in October, 1960, and Mr. Tacke was appointed to serve through October 1964.

Mr. Fratar is the Highway Division Contact Member on the Board of Direction, and Edmund J. Cantilli is Newsletter Editor.

Mr. Buckley was appointed Highway Division Representative on the Coordinating Committee on Transportation replacing Harmer Davis.

Joint Representation with Other Organizations

Highway Research Board: Ralph A. Moyer, Representative;
Roy E. Jorgenson, Alternate.

American Standards Association, Committee D-15: C. F. McCormack

This committee completed its initial assignment to prepare standards for measuring motor vehicle fleet accident experience and has submitted its recommendations to A. S. A. for review.

Task Committee on Terminals

Edward G. Wetzel (ch.). Committee now being formed.

Committee purpose: To define the purpose and scope of a permanent committee on the Division on highway transportation terminals.

Committee on Cooperation With Local Sections

Curtis J. Hooper (ch.), W. J. Mulder, Harold B. Britton, W. J. Miller, J. P. Ambler, F. R. McComb, Ellis Danner, W. O. Snyder, R. J. Paquette, R. M. Gillis, J. E. Havenner, D. L. Sargent, E. C. Simpson, J. W. Courter, S. C. Palmer.

The new chairman, Mr. Hooper, took over this committee activity upon the resignation of Mr. S. E. Ridge, chairman since the beginning of the committee. Other changes in the committee are shown by the listing above. The committee would appreciate having suggestions for activities for the coming year.

Committee on Highway Division Publications

The committee handled 27 papers during the year. Seventeen of these have been published in the Journal, seven were returned to the authors as not filling Society requirements and three are in the process of review.

This report represents a decline in the number of papers handled by the committee. To study the problem and recommend a course of action the chairman plans to meet with the Executive Committee and hold a meeting of the Publications Committee, in the near future.

Committee on Session Programs

Membership of this committee is composed of the chairman and secretary of the Executive Committee and the chairman of highway sessions for Society meetings. Membership for 1960 is: J. O. Morton, C. F. McCormack, Louis Duclos (New Orleans, March 1960), George Langsner (Reno, June 1960) and E. F. Copell (Boston, October 1960).

Membership for 1961, besides the Chairman and secretary will include Earle V. Miller (Phoenix, April 1961).

The committee as such has not held a meeting during the past year. However, in their active cooperation with the Executive Committee in preparation for the Society meetings, members have made outstanding contributions to the Society.

Committee on Developments in Highway Engineering and Construction

The committee has held no meeting during the year but has submitted a number of papers to the Society for consideration for publication. One of these, by W. E. Dean of Florida, appeared recently in Civil Engineering.

The chairman, Mr. E. M. Johnson, of Mississippi, feels he should not accept reappointment to the committee. His work has been most helpful.

Committee on Highway Drainage Structures

This committee held its organizational meeting in St. Paul in April, 1960. Two accomplishments resulted; one, compilation of a questionaire to all highway departments to determine the status of highway drainage practices and the second, initiation of a bibliography of selected research within the field. As a result of these two activities the committee will have information for decision on the areas the committee can best work in. The committee plans to meet in December in San Francisco to evaluate the results of the questionnaire.

Committee on Highway Planning and Finance

The chairman of this committee, Mr. Lynch, is retiring from committee activity this year, since he is also retiring from the Bureau of Public Roads. The committee reports a very active year with two committee meetings, one in Washington and one in New Orleans, together with sponsorship of joint sessions at New Orleans and Boston.

In the two years of its life, the committee has produced five papers for publications in the Journal. Two have been published, one was declined because a similar paper by the author had appeared in another magazine, one is at Society headquarters and the fifth is in the hands of the committee for review.

The completed report of the survey of engineering school curricula was distributed to the 126 institutions covered in the survey. Society headquarters has a copy of the report but it has not been published.

Committee on Traffic Engineering

The committee has maintained close liason with the Committee on Urban Transportation and the City Planning Division. Through the position and activities of the chairman, the committee has cooperated in areas of mutual interest with the officers and directors of the Institute of Traffic Engineers.

Committee on Urban Transportation

The committee feels that the time has come to hold a special conference devoted entirely to urban transportation. The committee is desirous of working with other interested committees of the Society in preparing for such a conference to be held sometime in the Fall of 1961 or early Spring of 1962. A committee meeting during the Boston Convention in October discussed further action on this.

WORLD TRAFFIC ENGINEERING CONFERENCE - 1961

A World Traffic Engineering Conference is to be held in Washington in August 1961. The announcement was made by the Joint Committee on International Weeks for Traffic Study in London.

The Conference will combine the 31st Annual Meeting of the Institute of Traffic Engineers and the 6th International Study Week in Traffic Engineering according to the announcement. It will be held during the week of August 21 to August 26, 1961, with the first three days devoted to the Annual Meeting of the Institute, and the next three days assugned to the International phase.

The programs of the two meetings will be integrated, with a large overlapping attendance anticipated. Specific themes tentatively scheduled include discussions of the following:

Objectives in the Development of Metropolitan Areas; Urban Transportation and Its Future; Traffic Engineering Research and Its Promotion by International Cooperation; Design of Rural Freeway Interchanges; Highway Traffic Accident Research; Electronics in Traffic Control; Freeway Traffic Operations and Control.

Speakers will be chosen on an international basis, to provide the best and widest coverage of the subjects. An opportunity will be provided in each session for a full discussion by all groups represented.

Following the week in Washington, a tour is planned for the foreign visitors to enable them to see at first-hand some of the results of U. S. traffic engineering work. The tentative itinerary includes, besides the Nation's capital, Baltimore, Philadelphia, Trenton, New York City and New Haven together with the highways linking these cities.

Attendance by approximately 100 European representatives is anticipated, together with delegates from all parts of the Western Hemisphere as well as other parts of the world. It is anticipated that a recent extension of the 1958 agreement with Russia for the exchange of scientific personnel will bring representation from that country. All traffic engineers in the world will be welcome, and a total attendance of nearly 1,000 persons is expected, with the greatest number from the U. S. and Canada.

The Joint Committee on International Weeks for Traffic Study is composed of representatives of three organizations: the World Touring and Automobile Organization, the Permanent International Association or Road Congresses, and the International Road Federation. It has sponsored five Traffic Study Weeks in Europe, the latest in Nice, France, during the week of September 19, 1960. The Washington meeting will serve as the sixth in the series.

The London contact is M. H. Perlowski, the Secretary of the Joint Committee, whose address is World Touring and Automobile Organization, 32 Chesham Place, London S. W. 1, England. The Washington contact is David M. Baldwin, Executive Secretary of the Institute of Traffic Engineers, 2029 K Street, N. W., Washington 6, D.C.

INTERNATIONAL CONFERENCE ON ROAD RESEARCH

Delegates from thirteen member and associated countries of the Organization for European Economic Co-operation have recommended that an international body should be set up to foster road safety research. This was the major decision taken at a four-day international meeting at the U. K. Road Research Laboratory of D.S.I.R. at Langley, Buckinghamshire.

It is suggested that the international body should be responsible for co-ordinating the research effort of the various countries concerned; to foster the interchange of road research information; and to recommend new researches on problems whose solution requires or would profit from international co-operation.

Sir William Glanville, C. B., C.B.E., F.R.S., U.K. Director of Road Research, was Chairman throughout the meeting, which was held under the auspices of the European Productivity Agency of O.E.E.C. The countries represented, in addition to the United Kingdom and the United States, were Austria, Belgium, Denmark, Spain, France, Greece, Italy, Norway, the Netherlands, Sweden and Switzerland.

SIGNS IMPAIRING SAFETY CAN BE BARRED

The Appellate Division of the New York Supreme Court has ruled that billboards and other advertising signs that impair traffic safety may be barred from private land along the highways. The restriction applies to signs that can be seen from the highway by a person with normal vision. The majority opinion noted that the Public Works superintendent had the authority to improve highway safety.

PERTINENT PUBLICATIONS

Highway Research Board Publications

"Traffic Behavior on Freeways." Bull 235 (1960) Pub. 698. 132 p., $2.40.
"Highway Laws, 1959." (1960) Bull 237 Pub. 700, 29 p., $0.60.
240. Highway Accident Studies. (1960) Pub. 726, 56 p., $1.00.
244. Effects of Traffic Control Devices. (1960) Pub. 730, 97 p., $1.80.
247. Sign Supports: Foundation Design. (1960) Pub. 733, 35 p., $0.80.
249. Highway Needs and Programming Priorities. (1960) Pub. 738, 75., $1.80.
252. Snow and Ice Control with Chemicals and Abrasives $.80.

SPECIAL REPORT

52. A Framework for Urban Studies: An Analysis of Urban-Metropolitan Development and Research Needs. (1960) Pub. 722, 29 p., $1.20.
53. State Highway Organization Charts, 1959 Revised. (1960) Pub. 723, 50 p., $1.60.
55. Highway Research in the United States. (1960) Pub. 736, 119 p., $2.80.

OTHER SERIES

Highway Research Board Publication Index 1956-1959. (1960) 129 p., $2.60.
Roadside Development, 1960. (1960) 25 p., $1.00.

(Checks or money orders may be made payable to "Highway Research Board" or to "National Academy of Sciences.") Address orders to: Highway Research Board, National Academy of Sciences, 2101 Constitution Ave., N. W., Washington 25, D. C.

A STUDY OF THE GENERATION OF PERSON TRIPS BY AREAS IN THE CENTRAL BUSINESS DISTRICT

By B. C. S. Harper and H. M. Edwards, Department of Civil Engineering, Queens University, Kingston, Ontario. May 1960. 31 pp. plus appendices.

This report presents the results of an investigation which endeavored to find out if the traffic that flows in and out of a city's center each day is directly related to the buildings, and hence the floor space in the area. It is believed such a relationship could be used in conjunction with sound economic forecasts of CBD space usage to predict future travel to the center more accurately than by existing techniques.

Use was made of the results of origin-destination traffic surveys and central business district floor space studies to develop regression equations for a number of cities. These equations explain the variations in number of person trips with destination in the CBD zones, on the basis of variation in the amount of floor space in three broad use classifications in the various zones - Retail, Service-Office, and Manufacturing-Warehousing.

ROADS AND THEIR TRAFFIC

Edited by Ernest Davies, Assoc. Inst. T., Blackie and Son Limited, London, England. June 1960. 350 pp., 35s.

This book is designed to inform the layman of the functions and scope of traffic engineering. Mr. Davies and 12 other contributors have prepared chapters for this book dealing with practical applications of traffic engineering, highways both in Great Britain and on the Continent, parking, traffic control, lighting and several other related topics. Each of the contributors is a specialist in his field. The list includes an American professor and several authors from continental Europe as well as England.

MEETING CALENDAR

ASCE MEETINGS - 1961

April 10-15	Phoenix Convention Contact: E. V. Miller Johanessen, Girard & Miller Phoenix, Arizona
October 16-20	Annual Convention, New York Hotel Statler
	1962
February 19-23	Houston Convention Hotel Shamrock, Houston, Texas

NON-ASCE MEETINGS - 1961

January 9-13	Highway Research Board Annual Meeting, Sheraton-Park Hotel, Washington, D. C. Contact: HRB, 2101 Constitution Ave., Washington 25, D. C.

March 28-30	Western Safety Congress Ambassador Hotel, Los Angeles, California, Contact: Greater LA Chapter, National Safety Council 3338 W. 8th St., Los Angeles 5, California
August 21-26	World Traffic Engineering Conference 31st Annual Meeting - Institute of Traffic Engineers and 6th International Study Week in Traffic Engineering Sheraton-Park Hotel, Washington, D. C. Contact: ITE, 2029 K St., N. W. Washington 6, D.C.

DEADLINE FOR MARCH 1961 NEWSLETTER

January 15, 1961

Send contributions to the Newsletter Editor:

EDMUND J. CANTILLI
Room 1202
The Port of New York Authority
111 Eighth Avenue
New York 11, New York

ASCE-WPCF JOINT SEWER MANUAL ISSUED

A major addition to the ASCE series of Manuals of Engineering Practice is now available in a new volume entitled "Design and Construction of Sanitary and Storm Sewers." Identified as No. 37, this publication is the result of several years of joint effort by the Sanitary Engineering Division of ASCE and the Walter Pollution Control Federation (formerly the Federation of Sewage and Industrial Wastes Associations).

The twelve-chapter sewer manual contains 283 pages, over 100 illustrations, 24 tables, and more than 100 references. As the first extended collection of information on the subject, it will make a valuable reference in an important phase of wastewater technology. Individual subjects covered include organization and administration of sewer projects, surveys and investigations, quantity of sanitary sewage and storm water, hydraulics of sewers, design of sewer systems, appurtenances and special structures, materials for sewer construction, structural requirements, construction plans and specifications, construction methods, and pumping stations.

The manual may be ordered with the coupon herewith. The list price is \$7.00 per copy. However, ASCE members may order the manual for \$3.50 per copy. The price to members of the Water Pollution Control Federation is the same upon application to their organization.

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NEW DIRECTORY IS AVAILABLE TO MEMBERS

The 1960 Directory is now available to members on request. The Directory lists the entire membership of the Society, giving the membership grade, position, and mailing address of each. In addition, there is a complete listing of the Honorary Members, past and present, and the Life Members. A useful geographical listing of the members is also included.

It goes without saying that the information contained in the Directory is of value to every member, and every member can obtain this valuable information. To receive your free copy of the Directory simply fill out the coupon below. Prompt delivery depends on prompt return of the coupon.

The Society publishes the membership Directory every other year. The next edition will be issued in 1962.

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PAPERS FROM THE 2nd CONFERENCE ON ELECTRONIC COMPUTATION

The papers presented at the 2nd Conference on Electronic Computation in Pittsburgh, September 8-9, 1960, are being offered in a single hard-bound volume. This special edition is composed of the thirty two technical papers, the welcome address, keynote address and three luncheon addresses, all as delivered at the Conference. The price (post-paid) is as follows:

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